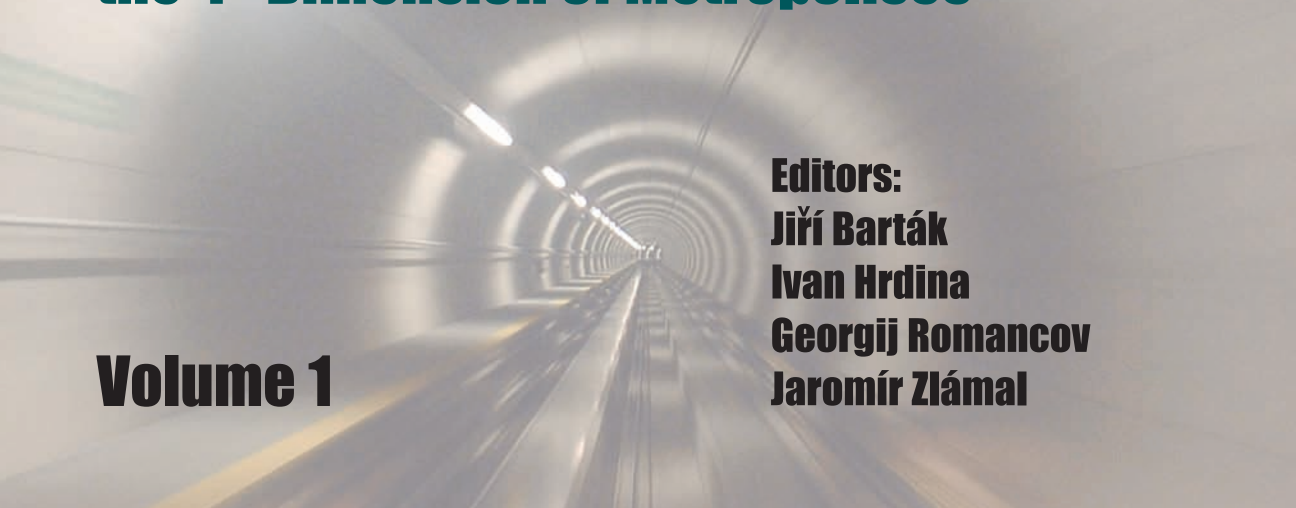
Underground Space

the 4th Dimension of Metropolises



Volume 1

Editors:
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Georgij Romancov
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UNDERGROUND SPACE – THE 4th DIMENSION OF METROPOLISES

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Underground Space – the 4th Dimension of Metropolises

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Preface

The Czech Tunnelling Committee ITA/AITES – as organizer of the 33rd World Tunnel Congress 2007 in Prague – has been elected by the ITA member countries three years ago. We have been very honoured by their confidence and grateful to the ITA for its support.

The ITA annual congresses and meetings have become premier events in the international tunnelling and underground construction calendar.

The theme of the WTC 2007 "Underground Space – the 4th Dimension of Metropolises" has been chosen as we feel it to be still more and more topical regarding the permanent growth of concentration of the world population into metropolises and megacities. The underground offers the hope of help to solving of related problems.

Hundreds of potential participants and interested companies and institutions from all over the world had been invited to submit their papers and contribute to the development of underground construction. The response to free paper submission was very impressive and many of the accepted papers contained new information and demonstrated further growth in the significance of our industrial branch. We would like to reaffirm our appreciation to all our colleagues for their great willingness to share knowledge and for their continued support.

The Proceedings of the ITA-AITES WTC 2007 – the book and the CD-ROM – present the full papers accepted by the Scientific Council of the Congress. Authors submitted 382 abstracts which were reviewed and 316 full papers were selected for publication. The published papers have been grouped into 9 sections – main discussion topics of the Congress.

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The editors would like to acknowledge the great effort expended by all these colleagues and friends in the organization of the ITA-AITES 2007 Congress in Prague.
Jiří Barták – Ivan Hrdina – Georgij Romancov – Jaromír Zlámal

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Section 1
Underground city design planning of underground constructions

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Largest transportation system – Metro and road in the same tunnel

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ABSTRACT: Brief description of design and construction features of a unique project, Serebrianoborsky tunnels, is presented in the article. The project comprises three tunnels, each 1514 m long, including two 14.2 m diameter transport tunnels, one 6.3 m diameter service tunnel and five cross drifts connecting these tunnels. The tunnels are built using Herrenknecht TBMs with shields of respective diameters operating according to the bentonite face hydraulic support technology. The design was awarded prize of International Exhibition of Innovations in Brussels.

In accordance with the new Moscow Master Plan, extensive trunk street/road development program is underway in the city.

A trend seems to evolve for new trunk roads to be laid in tunnels crossing the whole city districts and leaving intact both the habitat and surface buildings, man-made structures and architectural monuments; it means that there appear long trunk road urban tunnels. This is particularly important for such a complex urban body as Moscow where the historically formed ring/radial trunk street system makes it insistent to create a transportation system capable of providing for both perfect traffic control and links between peripheral districts and the centre.

On the basis of the Master Plan, a new radial transportation corridor, Krasnopresnenski Avenue, is being formed in Moscow, the corridor making it possible both to create a new trunk road in the western sector of the capital and to provide conditions for road users to comfortably pass from the city center to the “Baltiya” Moscow – Riga Federal Highway.

The first “Baltiya” Moscow – Riga Federal Highway section joining the Moscow Ring Road (MKAD) was completed in 1996. However, the highway could not enter the city owing to the fact that the alignment crossed two protection zones: the Serebryany Bor forest park and the Krylatskoye dwelling and nature territory. Construction of a surface road could infringe ecological situation of the region since it would have been inevitable to cut trees in the Serebriany Bor forest, to demolish some important dwelling buildings and to disturb natural landscape.

At present, the Moscow – Riga trunk road is linked with the city centre through MKAD and its interchanges at Roubliovskoye and Volokolamskoye Chaussees resulting in essential superfluous run and additional traffic load at MKAD section between these interchanges.

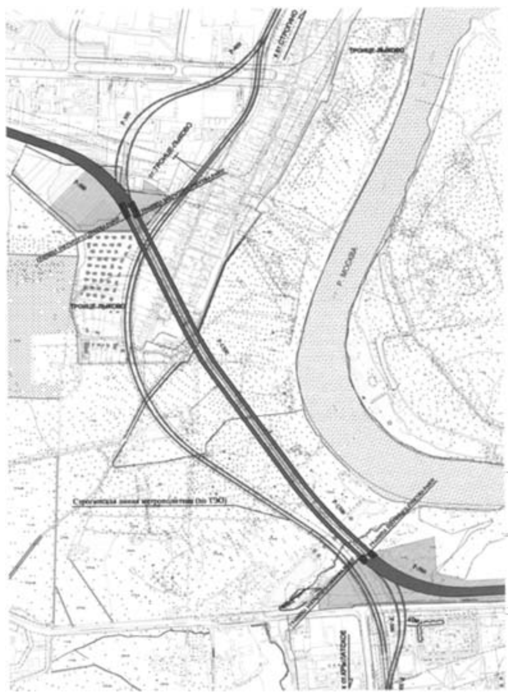

Figure 1. Tunnel site with approaches combined with Stroguinskaya metro line.

Besides, the Stroguinskaya metro line from Krylatskoye Station to Stroguino Station linking household areas of this region with the city centre is currently under construction along an alignment close to that of the proposed tunnels (Figure 1).

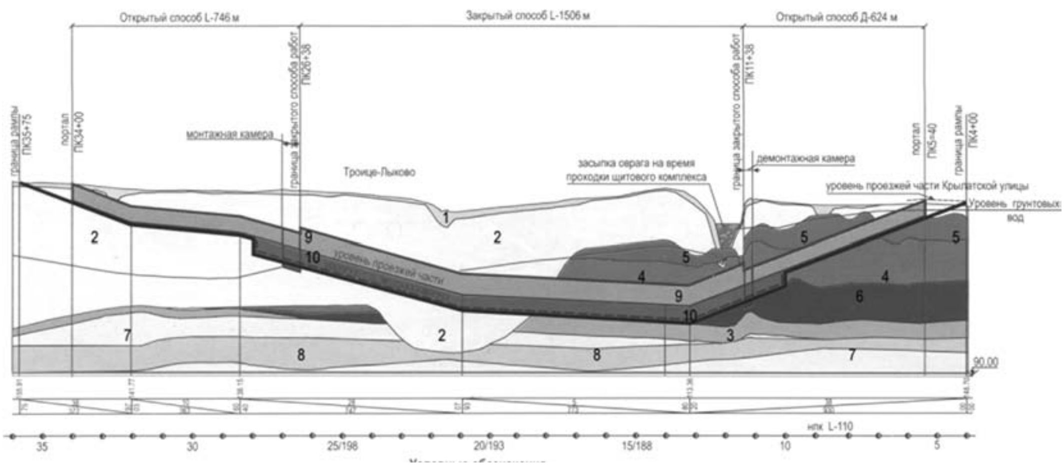

Figure 2. Longitudinal profile of tunnel section with approaches combined with Stroguinskaya metro line. 1 – filled-in soils: sand, sandy-loam, unconsolidated loam with brick and concrete breakage; 2 – wet and water-saturated alluvial and aqua-glacial sand of various grain size with layers of sandy-loam and loam, locally with gravel and boulders; 3 – tough-plastic and semi-hard (occasionally hard) Jurassic clay; 4 – semi-hard (occasionally hard) Jurassic clay; 5 – water-saturated chalk sand and sandy-loam, soft loam with sandy-loam layers; 6 – tough-plastic and semi-hard Jurassic loam, water-saturated sandy-loam; 7 – medium-hard and hard fractured and badly fractured water-saturated coal limestone; 8 – hard and semi-hard coal clay and clayey marl; 9 – road right-of-way; 10 – metro right-of-way.

Thus, serious problems that arose in the prestigious North-West region of Moscow were to be urgently solved to improve transportation conditions of the city.

Metroguiprotrans designers could find the solution having proposed basically new engineering and construction approach consisting in laying two 14.2 m diameter tunnel barrels under the whole protection and household zone, each tunnel accommodating two transportation modes: road for 3 traffic lanes in the upper level and metro tracks in the lower one. This approach was patented and awarded with a prize of International Exhibition of Innovations held in Brussels in 2003; at present, such world-first “dual-purpose” tunnel barrels are being built at the section where alignments of Krasnopresnenski Avenue and Stroguinskaya line of Moscow Metro coincide.

Construction site is located at the second and the third right flood land terraces of the Moskva River, absolute elevations of the graded surface varying from 145 to 155 m.

Geological composition of the region is represented with deposits of the Quaternary, Cretaceous, Jurassic and Coal periods.

The Quaternary deposits encountered at many sites are represented with technogenic as well as with alluvial and aqua-glacial soils, presumably those of sandy and sandy-loam types.

There are several ground water tables along the alignment including Jurassic, Ratmir and Souvorov water-bearing layers.

The free ground water table is measured at absolute elevations from 126 to 130 m. Thickness of the layers vary from 12 to 30 m. Permeability factor varies from 4 to 15 m/day (Figure 2).

Implementation of this project has become possible owing to the fact that principally new Herrenknecht tunnel boring machines were used, the machines operating with combined hydraulic and air face support 14.2 m diameter shields at depths providing for complete safety of the city habitat independently of geological and hydrological conditions. It made possible to successfully complete driving all Krasnopresnenski Avenue tunnels (Figure 3).

The Krasnopresnenski Avenue tunnel section thus consists of three tunnels – two 14 m diameter 1514 m long transport tunnels and one 6 m diameter 1514 m long service tunnel located between the transport ones (Figure 4). The service tunnel is used to accommodate utilities, to perform ventilation functions and, in case of emergency, to provide for evacuation of the passengers and for unhampered approach of fire and rescue teams to the site of emergency. Cross drifts connecting the three tunnels are made every 250 m to provide for the approach. Total length of the transport tunnels including the ramp sections makes 2550 m, two 1514 m long tunnels having been driven in closed techniques using one tunnel boring machine. The following construction sequence was used: first the left transport tunnel, then the service tunnel (while driving the service tunnel, the main TBM was dismantled, transported to the base ground and reassembled to drive the

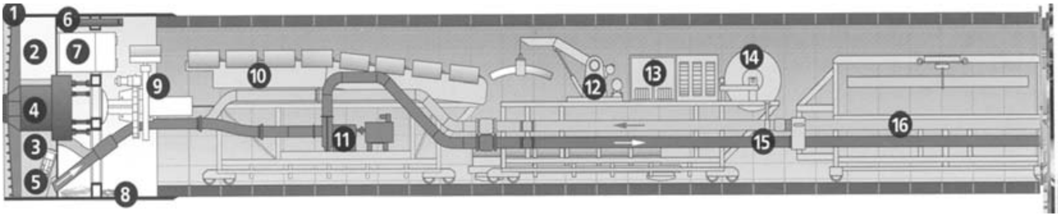

Figure 3. Mixshield design. 1 – cutterhead; 2 – air cushion; 3 – bentonite suspension; 4 – drive; 5 – stone crusher; 6 – cylinder; 7 – decompression chamber; 8 – navigation cylinder; 9 – segment erector; 10 – segment transporter; 11 – muck pump; 12 – segment lifting crane; 13 – power shield; 14 – cable drum; 15 – muck removal pipeline; 16 – feeding pipeline.

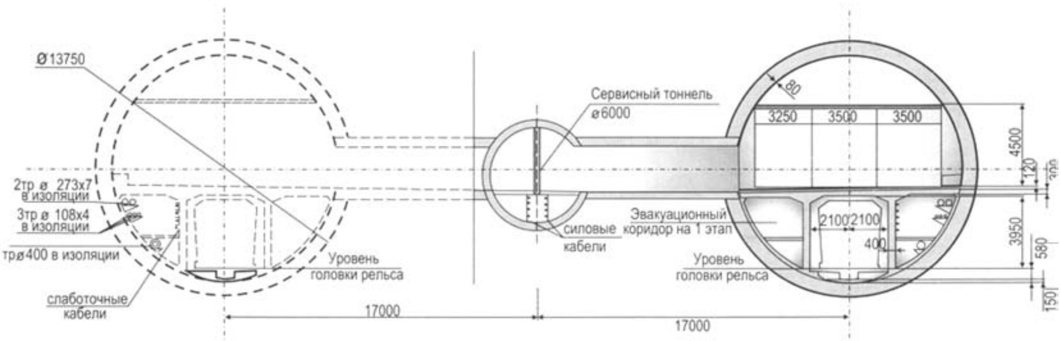

Figure 4. Cross-sections of Serebryanoborsky Tunnels.

right transport tunnel); the right tunnel was the last to be driven. In order to provide for the tunnels to be built, a 9.2 m^2 base ground was arranged with start shafts for all the tunnels, building for all separation equipment, storage ground for lining segments, compressor room and utilities.

In order to be able to drive all three tunnels from the same base ground, a 6.3 m diameter TBM of the same type as TBM for the main tunnels. Reinforced concrete segments for lining rings were manufactured at an updated plant in Moscow in accordance with design prepared by Metroguiprotrans OJSC specialists. Rings for the main tunnels are as wide as 2 m, each consisting of 8 main segments and a locking one.

The left tunnel driving commenced in May 2004. At present, all the three tunnels are driven, while construction of the cross drifts and the roadway as well as installation of technological equipment is underway. Average and maximum advance rates of the main TBM made 200 m and 250 m per month respectively. Owing to strict adherence to the design face support hydraulic pressure values, surface settlement in the course of driving were brought to minimum and did not exceed 10–12 mm.

At two thirds of the alignment, the tunnels pass through water-saturated sand soils and through clay soils at the remaining third. Given the difficult ground conditions, the cross drifts are built using special soil stabilization forestalling methods. The techniques used for each drift differ depending on particular soil properties. Horizontal jet-grouting, cement grouting, stabilization with various materials and soil freezing are used.

In order to provide for the required air quality and safety parameters in the tunnels in case of fire, forced feeding and extraction ventilation and powerful smoke-removal systems are installed. Each tunnel is ventilated with respective individual systems. The tunnels are built according to the highest fire-safety requirements; the whole inner surface of each tunnel is protected with special cover capable of withstanding open fire action within 3 hours.

Completion of this tremendous project scheduled for December 2007 will finalize creation of the direct transport communication between the "Baltiya" expressway and the centre of Moscow resulting in mitigation for the traffic-overloaded Volokolamskoye and Roubliovskoye Chaussees. Besides, 1.5 km long metro line will be built.

Tunneling practice will get enriched with one of the world most important underground structures, a complex of combined road and metro systems making it possible for major cities to more efficiently solve their transportation problems.

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Anchor recovery under extreme conditions in downtown Leipzig, Germany

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ABSTRACT: In the inner city of Leipzig, Germany, the London, U.K. based AMEC group, an international infrastructure specialist, faced the problem that old cable anchors from the construction pit of a multiplex shopping centre built by AMEC in 1999/2000 had to be recovered from under existing buildings. The anchors were finally recovered by a pile borer and a slurry wall excavator in a first stage and by pipe jacking under compressed air along the anchor axes in a second stage.

1 INTRODUCTION

In the years 1999 and 2000 the British AMEC group, represented by its Leipzig office, built and afterwards managed the multiplex centre “Petersbogen”, which is in the heart of the inner city of Leipzig. The centre is comprised of shops, restaurants, a cinema, a casino and the judicial branch of the Leipzig University. The complex is four (4) stories high and has four (4) basement levels of which the first is a shopping area, whereas the others are used for underground parking and building utilities.

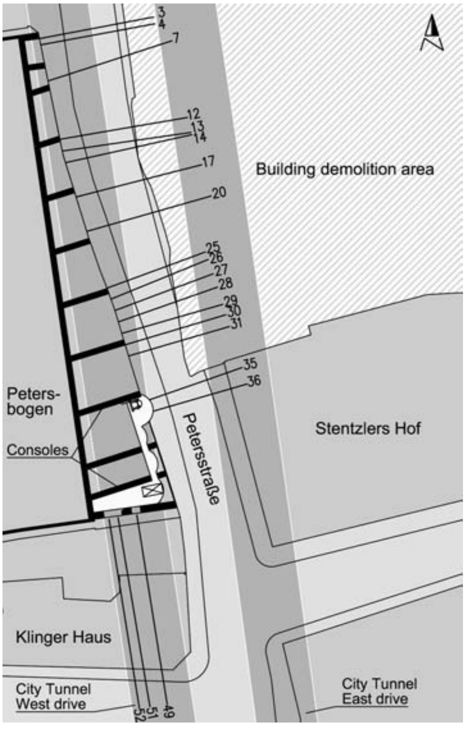

Figure 1. General layout.

During the design phase of the building AMEC had to consider that a future underground railway line, the so-called “City Tunnel Leipzig” consisting of two tubes, would pass in a shallow alignment under the property of the Petersbogen’s southeast corner. As part of the building permit AMEC had to guarantee that no permanent installations would be positioned within the future tunnel cross sections.

Due to the high ground water level with permeable layers of soil on top of impermeable ones, a water tight construction pit was planned, enclosed by deep slurry walls reaching into the impermeable layers. In the slurry walls, steel beams were positioned to stabilise the free standing part of the wall. In order to reduce wall movement and the steel beam cross section two layers of cable anchors were planned by the designer. The anchor geometry accounted for limitations from the tunnel.

2 THE TROUBLE STARTED

In 1999, the contractor for the construction pit began the work as designed and tendered. Soon after the work began he ran into remnants of foundation from former buildings, which had been announced in the tender documents. Due to excavation problems the contractor changed the building pit design to three rows of anchors. But now the bottom layer was ending right in the future City Tunnel cross section.

The contractor promised to get the anchors back out of the ground by explosive recovery. This meant small ductwork would be interlaced within the cables of the anchors, so that the grout bulb could be loaded with explosives in the form of a cortex. Through the explosion, the grout bulb would then be cracked, making the steel cables easy to pull out of the ground. Also, a back-up method was offered.

The work continued according to the alternative plan. Unfortunately, when time came to recover the anchors, the contractor found that he could not fill some of the ductworks with any cortex and others could only be filled with a smaller and less effective cortex than planned. The result was that the contractor failed to recover 43% of the anchors. As if that wasn't enough, the alternative recovery method with the hollow drill rods also failed.

The contractor tried several other recovery methods. First, he tried to loosen the bedding of the anchors in the soil with the help of jet grouting lances. After that, he tried to destroy the grout bulbs with large amount of explosives which were detonated in extra boreholes bored parallel to the anchors. Even the pulling technique was varied from pulling all cables of an anchor at once to pulling only single cables or smaller groups. But all methods failed.

Finally, the construction schedule became so pressing on AMEC that the regular construction works had to continue.

3 A NEW ATTEMPT

At this stage AMEC requested Babendererde Ingenieure to prepare new solutions for the anchor recovery. As specialists for TBM tunnelling, the first solution was to check whether the slurry TBMs of the City Tunnel could simply drive through the cable anchors. This idea was quickly abolished due to unpredictable high risks and costs.

It was then investigated what would be necessary to recover the anchors from inside of the TBMs. Although it was technically feasible, the costs were enormous and unacceptable for AMEC.

The third alternative was to construct small, hand driven ducts, secured with fibre reinforced shotcrete under compressed air. Starting from within the basement level of the Petersbogen, miners would have to dig a tunnel towards the critical zone of the anchors and then, from one anchor to the next, to cut away the sections within the City Tunnel alignment. The shotcrete ducts would have been no match for the TBMs, which were equipped for mixed face conditions comprising cohesive and non cohesive soils with stones and layers of hard, abrasive quartzite.

After the preparation of this solution the client suddenly learned that a building opposite to the Petersbogen, under which 15 of the 20 remaining anchors were lying, was about to be demolished to make space for a new department store.

New options were considered. The final solution was to pull the anchors out of the ground with the help

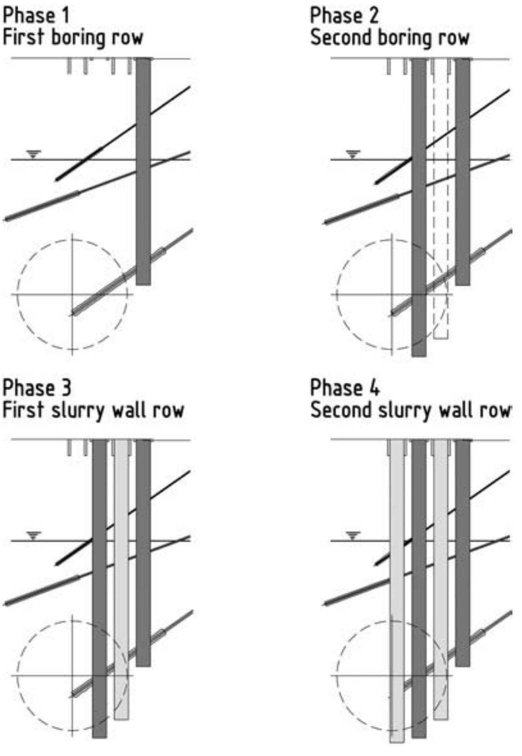

Figure 2. Working steps for recovery from above ground.

of pile borers and slurry wall excavators which were already available due to required site preparations for the new department store. So in the summer 2004 a 75t pile borer and a 120t slurry wall excavator were driven to Leipzig to dig for cable anchor pieces with a maximum weight of 30 kg.

4 GOING FISHING

The final plan was to cut the anchors with a pile borer ID 1,2 m. Two (2) rows of cuts were foreseen. The first row was supposed to cut each anchor into two (2) sections, one (1) within the tunnel and one outside of the tunnel. The second row was supposed to cut the sections within the tunnel into two (2) sections of equal size ranging between 3 and 4 m in average length. The bore holes were to be filled with a one-phase boring suspension which hardened after a while to form an artificial soil of similar strength compared to the natural soil in this area.

Then the slurry wall excavator would perform two (2) rows of excavation to pull out the remaining sections of the cable anchors. The trenches were to be stabilized by the same suspension which was already being used for the borings.

Figure 3. Construction site.

5 RECOVERY FROM ABOVE

The excavation works were no problem for the pile borer and the trench excavator. The pile borer cut even through anchors with nine (9) steel cables without any recognisable evidence for the driver. Thus, every single bucket of excavated material from the ground had to be closely observed while pulled above ground and dumped into the bucket of waiting wheel loader. Often cables of an anchor were obviously hanging in the teeth of the boring bucket. In general, the cable sections were between 60 and 120 cm long and heavily twisted because of the boring rotation. Yet some cable sections also were squeezed into the bucket and were spotted during the dumping onto the wheel loader bucket. The emptying of the wheel loader was also closely observed. Some cables were disintegrated to single wires and it was often only during the last dumping from the wheel loader that such wires were found.

Time and again a “hit” on an anchor was indicated by steel cable sections hanging from the boring bucket. Then, in the next one or two buckets coming up, more pieces from that hit were found. Nevertheless, it also happened that just a few distorted wires were the only indication of a successful hit.

The important task of the Supervising Engineer in this stage was to explicitly map, record and photograph every single finding, and then identify which anchor was cut. The mapping was used to get a closer idea where exactly each anchor was lying to verify the positions of the following slurry wall excavations. If one considers that there were two (2) more layers of cable anchors above the one searched for and that through the deviations of the anchors and the borings alike sometimes anchors were found where none had been expected, the identification became a big puzzle which finally was solved.

After all findings had been interpreted, the excavation with the slurry wall excavator began. Whilst watching and searching through the excavated material during the borings was often tedious work, now the big hits were being made. As planned, the trench excavator easily pulled out the remaining sections of the cable anchors.

Up to 4 m long sections were brought to the surface in one piece. As the locations of the anchors were quite well known through the borings, it could even be predicted at which position and depth anchors should be found. The job advanced well and was finished within the tight and unmodifiable time frame which was given to AMEC by their neighbour and his construction works.

Figure 4. Anchor section under slurry wall excavator.

Figure 5. Recovered and cleaned anchor section.

6 FIVE TO GO

After the successful recovery of the first 15 anchors there were still five (5) to get out. All five (5) anchors were located in the southeast corner of the shopping centre. Anchor no. 35 and 36 extended to the east. They were 19 m long, bored at an angle of 35° and had a 9.5 m long grout bulb each. The anchors were perpendicular to the City Tunnel axis and the last approximately 4.5 m of the grout bulbs were directly within City Tunnel east drive cross section. The intersection between the anchors and the tunnel was under one of the neighbour's buildings, the so-called "Stenzlers Hof".

The other three anchors no. 49, 51 and 52 extended to the south. They were 32 m long, bored at an angle of 35° as well and had a 11.5 m long grout bulb each. The anchors were parallel to the City Tunnel west drive crossing it from the top to the bottom at a length of about 15 m. Due to their length, the anchors' grout bulb started shortly under the tunnel's invert. Also in this case the intersection between the anchors and the tunnel was situated under a listed building, the so-called "Klinger Haus".

Again, as previously mentioned, a recovery from above was impossible as the important sections of the anchors were covered by buildings. So the designer, as well as the client, accepted that the only solution left was to recover the anchors from underground, although it was unclear at this moment how, and from where, this should be done. The deepest parts of the anchors to recover were about 25 m below the surface and 17 m below the ground water table.

Digging down from the Petersstraße was impossible. It is the main shopping street in Leipzig. The compensation alone to shop owners due to construction work in front of their shops would have been enormous. In addition, any work from within the basements of the buildings above the anchors was impossible. The rooms were too small for any sensible operation and they were occupied by the tenants. Therefore, the only possible solution was to start from somewhere within the property of the client, which was the Petersbogen building.

The anchor heads were lying roughly at a depth at the middle of the third basement level and 1 m below the ground water level. Fortunately, the construction of the building in the southeast corner was different from the rest of it. In general, the building had four basement levels. But in the area of the later City Tunnel the client had been allowed to build only two basement levels, thus the third and fourth basement levels had to be reduced in their expansion. In order to keep the future tunnel free from massive loads from the overhanging building part, it was constructed on huge reinforced concrete consoles which were attached to the side walls of the third and fourth basement levels.

As a result of the special construction, the five anchors ended actually under the Petersbogen building. Due to the fact we decided to cut an opening into the slab of the parking level in the southeast corner and dig tunnels under the cover of the slab to reach the anchor heads. Then micro tunnels could be launched, one for each anchor.

With the help of 1.2 m ID pipes driven under 35° downwards over each anchor, the anchors could be dug out and cut to pieces parallel to the jacking progress. The excavation had to be performed by hand, by a man sitting in the head of the pipe. Since pipes would end in the City Tunnel cross section, a pipe material had to be selected which later would mean no harm to the TBMs of the City Tunnel.

The decision was made to use CC-GRP (centrifugally cast glass fiber reinforced plastics) pipes made by HOBAS. A Slurry TBM with discs is able to cut through such pipes when it comes upon them in the ground. Nevertheless, any steel elements permanently installed in the pipes were forbidden.

For steering and as protection for the first pipe's face, a mini steel shield was necessary although it had to be dismantled and carried back out of the pipe of each tunnel drive.

The biggest challenge and impact during excavation was coming from the ground water. The ground water table was equal to the level of the slab. It was impossible to lower it by pumping with mainly sands to excavate through, buildings all around and the deepest anchor sections being at 17 m below the ground water table. Under the given boundary conditions, ground freezing was also impossible. Therefore the only option left was excavation under compressed air. A service room, in the very south-east corner of the second basement level, was chosen as the entry point to the tunnels.

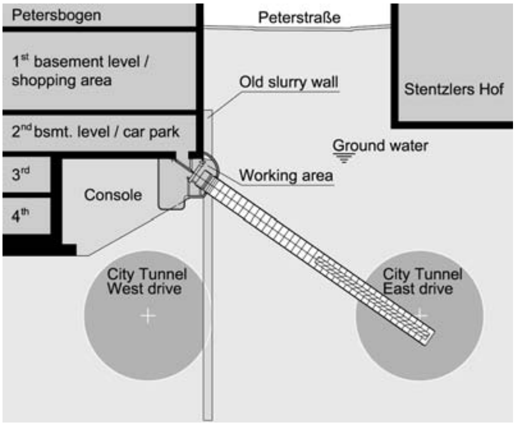

Figure 6. Recovery from underground.

The decision was made to cut a hole $3.0 \times 1.5\,\text{m}^2$ into the floor and excavate downwards from there. The service room walls consisted of reinforced concrete and it had just one door. The door had to be taken out and an air lock was connected to the opening. Static calculations confirmed that the walls would withstand 1.8 bar inner air pressure.

7 SITE INSTALLATION

After a short tender phase with a hand picked group of candidates site installation began in December 2005. The winning bidder Gildemeister GmbH & Co. KG from Berlin, Germany, started to alter the service room of the parking level. The hole was cut into the floor, a rented air lock was installed to the door opening and a serial of core drillings was set through the upper working chamber wall to enable installation of all the required utility lines needed in the working chamber. A rail system from the parking area through the air lock into the upper working chamber for hand pushed lorries and a rail crane under the ceiling completed the preparations in the building. Meanwhile outside of the building, the compressed air stations, a medical air lock, stores and accommodations were installed in an area next to the Petersbogen. This area was held free by AMEC for future expansions of the shopping centre.

8 RECOVERY FROM UNDERGROUND

In January 2006 the compressed air works began. The south working chamber along anchors 49, 51 and 52 was excavated between the first two consoles under complete cover of the slab of the second basement level. The east wall of the chamber, which was the only one open to the in-situ soil beneath the basement levels, was secured with an arch of reinforced shotcrete. Then openings were cut into the first console at the positions of anchors 49 to 52 in preparation of the pipe jacking. After that, an opening was cut into the second console to dig a tunnel towards anchors 35 and 36, where the north working chamber was excavated.

Figure 7. Air lock in underground car park.

Meanwhile, the contractor transported a custom made jacking frame to the position of anchor 52 and installed it between the two consoles. He then installed the mini-shield.

After a learning period, the overall excavation speed became faster than expected with exception of tunnelling through a quartzite layer, which was time consuming and which each of the five tunnels had to pass through. The pipe jacking got underway with 40 cm long pipes because longer pipes could not pass through the air lock. Soil transport was carried out with a small lorry going up and down the pipe. The sand and silt was taken from the face with a small blade. Hand-held core drills were used to drill holes into the quartzite rock and a hydraulic splitter was used to break it.

Parallel to the excavation, the anchor became visible more and more and it was regularly cut every 40 cm parallel to the laying of a new pipe.

After the intended depth of the tunnel was reached, the mini shield had to be dismantled and transported up to the working chamber again. The tunnel was cleared of all installations and finally filled with artificial soil. Afterwards, the jacking frame was moved to anchor 51

Figure 8. Jacking station for anchor 51.

Figure 9. Excavation in the head of the pipe.

Figure 10. Anchor in the face.

and modified to fit to the new position and then the working cycle started again. One after another, anchors 51, 49, 36 and 35 were recovered.

9 COMPRESSED AIR PROBLEMS

It was obvious that the air pressure had to be increased concurrent with the increasing depth of the excavation. It was discovered during the excavation of anchor 52 that losses of compressed air were much higher than expected and finally they exceeded the capacity of the compressor stations.

Extensive sealing efforts became necessary. The working chambers were sprayed completely on all surfaces with a water reacting polymer, forming a rubber-like inliner. The ring gap around the jacking pipes (the shield had a small over-cut) was filled with bentonite, and the pipe joints were sealed with a custom made rubber gasket. Together, these measures significantly stopped the air losses enough to continue excavation.

10 IMPACT ON THE NEIGHBOURHOOD

The excavation continued with hardly any affect on the neighbourhood given that the setup of the construction site was in an empty building pit, as well as in the basement of the Petersbogen. Only the compressed air was responsible for some excitement during the middle of the construction. It is normal when compressed air is pressed into the ground that it tries to rise to the surface. In open areas the air dissipates over a wide area, but in this case, in the heart of the city with only buildings and bitumen paved sidewalks and roads, the air concentrated around only the few escape points available. This time it was on a 50 m long strip along the curbstone of the Petersstraße. One night after some heavy rain, unexpectedly a lot of bubbles became visible. The police was called on the scene three (3) times in 24 hours (once, every shift) looking for gas leaks.

An additional point where the air exited was on the opposite side of the Petersstraße, in the basement of the Stentzlers Hof. The air was coming in through a joint in the foundation slab. Together with the air, a small amount of water was pressed in as the slab was only marginally above the ground water level. It took a few words of reassurance to convince the owner that there was no danger to his property.

11 END OF WORKS

Finally, by the end of May 2006, the last anchor was recovered and the working chambers were all filled with artificial soil. Not only had all anchors been recovered, but it was also revealed by in depth investigations of all anchor pieces that design and execution faults on the explosive recovery were the reason why that method failed for 43% of the anchors. The ductwork had been designed at the lower limit of the theoretical required cross section.

Nevertheless, it is the opinion of the authors that the explosive recovery is a capable method, which was demonstrated successfully at another construction site in Berlin, Germany, where 175 cable anchors were recovered. In Leipzig, if the design had simply chosen larger ductwork, accounting for the importance of the recovery, and the realisation that sometimes an increase of effort is necessary, explosive recovery could have worked as well.

12 CONCLUSIONS

In the inner City of Leipzig, Germany, 20 cable anchors had to be recovered. Dependent on the individual boundary conditions two different, innovative recovery methods were designed and successfully executed with hardly any influence on the public life around the construction sites: the recovery with pile borers and slurry wall excavators from above the ground and the excavation with the help of pipe jacking tunnels starting under an existing building. Both methods proved successful and it was even possible to find out why the intended recovery by explosions in the grout bulbs failed.

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Construction of Metro IV.C2 Ládví – Letňany in Prague

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ABSTRACT: The construction of another leg of Prague Metro's C line is currently in progress. The extension with the length totalling 3.9 km is designed as an underground Metro route with 3 stations in open foundation pits. The technology of building the sections between stations is subject to the geological composition of the area. Under the layer of made-up ground, argillaceous earths and arenaceous marls there are layers of sand rocks and clay stones, the characteristics of which allow building part of the sections between stations by tunnelling technologies. For the most favourable sections the project authors suggest driven dual-rail tunnels of a closed profile with the floor arch of the stope area of 58–66 sq. metres. For the other sections they suggest using single-rail tunnels or sections in open foundation pits. The conceptual design of the stations is based on the latest trends used in the construction of the section put into operation in 2004. The Stříškov station is designed as a single-nave hall. The bearing structure of the glass areas is designed from steel. The common hall area integrates the vestibule and platforms. The height of the station's interior consists of three functional levels, i.e. the platform level, the mezzanine level and the ambient ground level – the ground floor. All the three publicly accessible levels form a lockable area, which is only available to the public when the Metro operates. The Prosek station is designed as a classic monolithic reinforced-concrete building with three operational levels – the level below the platform, the platform and the vestibule (underground). The structure is designed with steel columns located between rails. The Letňany station is designed as a monolithic reinforced-concrete frame with a variable width of three plus one fields. The platform area features a simple frame with an opening for direct lighting. The station consists of three storeys, with two stories at the platform. A single-storey structure of a side-track building, with two rows of columns, is adjacent to the station.

1 INTRODUCTION

The underground is the main backbone system of the Prague transport skeleton. The new routes improve the transport serviceability of the localities, create new development areas and increase the value of areas around the new stations. The implemented extension of line C from the Ládví station onwards will bring about all these advantages. The transport situation will improve not only in Prosek and Letňany, but also in the whole north-eastern area of Prague. There will be a large bus terminus station at the Letňany station, and a park-and-ride yard (P + R), which is highly needed in this part of the city.

In relation to the completed 1st stage of the 4th operating section of underground line C (section Holešovice railway station – Ládví), the construction of its extension commenced in 2004 in the direction of Prosek and Letňany, with a long-term municipal transport terminal station in the area of the Prague exhibition ground in Letňany. The track is laid under the ground in a relatively shallow depth, the stations are performed by cut and covered method, the tunnels are partly driven and partly performed by cut and covered method and they are mostly double-tracked. The construction is located in Prague 8, 9 and 18, i.e. in the cadastral districts of Kobylisy, Libeň, Prosek, Střížkov, Vysočany and Letňany.

As every similar project of metro line, even this one includes a number of interesting structural solutions from the field of tunneling, concrete construction and architectural ideas that, according to our opinion, are worth attention.

The investor of the project is Dopravní podnik hl.m. Prahy a.s. (a public transport operator), represented by the contracting authority, Inženýring dopravních staveb a.s.

The building permit documentation, tender documentation and the detailed design documentation have been elaborated by METROPROJEKT Praha a.s.

2 CONSTRUCTION OF THE UNDERGROUND AS A PART OF THE PRAGUE TRANSPORT INFRASTRUCTURE DEVELOPMENT

2.1 *Principles of the transport-urban solution*

The track leads from the existing terminus Ládví to the Střížkov station, which will serve the locality of Prosek, and the remaining part of the locality will be served by the Prosek station. A temporary park-and-ride yard with 203 parking places is designed at the Střížkov station. At the new terminus in Letňany, which has two vestibules, a bus terminal station is designed with a large parking area and a park-and-ride yard with 683 parking places next to the northern vestibule. Both these vestibules meet requirements for future connecting to the planned Prague exhibition ground.

2.2 *Basic data on the construction*

Length of construction IV.C2 4599 m Maximum longitudinal gradient of the track 3,95% Minimum horizontal radius of the track rails 500 m Number of stations 3.

1. Střížkov – an excavated station with a side platform and a single vestibule incorporated into the station hall. The level of the platform is 5.6 m under the surface level,
2. Prosek – an excavated station with a side platform and a single underground vestibule connected to underpasses under Vysočanská and Prosecká Sts. The level of the platform is 10.6 m under the ground,
3. Letňany – a long-term terminus, excavated, with an island platform connected to 2 underground vestibules. The level of the platform is 10.3 m under the landscaped surrounding ground.

3 STRUCTURAL AND ARCHITECTONIC DESIGN OF THE STATIONS

3.1 *Střížkov station*

The Střížkov station is a wayside excavated station with side platforms, located at the crossroads of Vysočanská and Teplická Sts. Along with the track section in the direction of the Prosek station it represents a part of track IV.C2, where the underground track level is closest to the surface.

It is an open hall room and thank to a light steel structure of the roofing it enables maximum glazing of the peripheral jacket of the station and thus lightening of the station hall by daylight, which makes it look like an overground station, although both the platforms are deeper compared to the surroundings ground. By opening of one sidewall into the opened excavated atrium, the station is in a direct visual contact with the external space and enables future level connecting with a shopping centre connected directly to the atrium room. On the other side, the station is connected to an underpass under Vysočanská St., enabling comfortable disability access to both platform edges. The internal room of the public part of the station is vertically divided into three functional levels, i.e. the platform level, the counterlevel and the level corresponding to the surrounding ground. The operating-technical part of the station is located under the ground, the underpass under Vysočanská street and the structure of the intermediate ceiling over the platform including the flooring of the foot-bridges are made of cast-in-situ reinforced concrete.

The supporting structure of the glazed roof of the station consists of two slanted steel arches decussating each other with a box cross section of a changeable height. They are fixed in massive concrete foundation

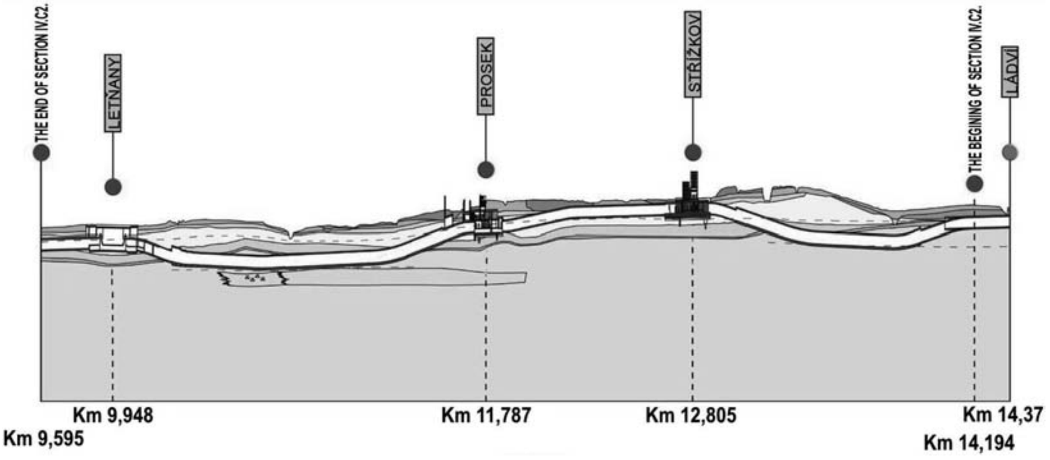

Figure 1. Longitudinal profile of metro IV.C2.

blocks hidden in the underground parts of the station. Four rows of tow bars suspended on the arches hold the roof structure. The main central box girder consists of two plain-web components, interconnected in the lower and upper parts with a framed girder. The girder is equipped with a number of transverse ribs – beams of changeable height. Ribs are interconnected by horizontal steel plate girder of circular shape on the side of the roof . The roof periphery is supported by steel socketed stanchions of "Y" shape with a changeable cross section. The glazed peripheral jacket is connected to these stanchions.

The total length of the Střížkov station structure is 228 m and the steel roofing structure with a ground plan shaped like a lens is 130 m long and its greatest width is 42 m.

3.2 *Prosek station*

The Prosek station is a wayside excavated station with side platforms, located at the crossroads of Prosecká and Vysočanská. Given a greater depth of the underground (the platform level is 10.6 m under the ground), the station is fully covered under the ground, with square glazes skylights designed in the steel structure in the area of the vestibule and the connecting shopping passages enabling partial lighting of the interior of the station by daylight. The street level is reached by the

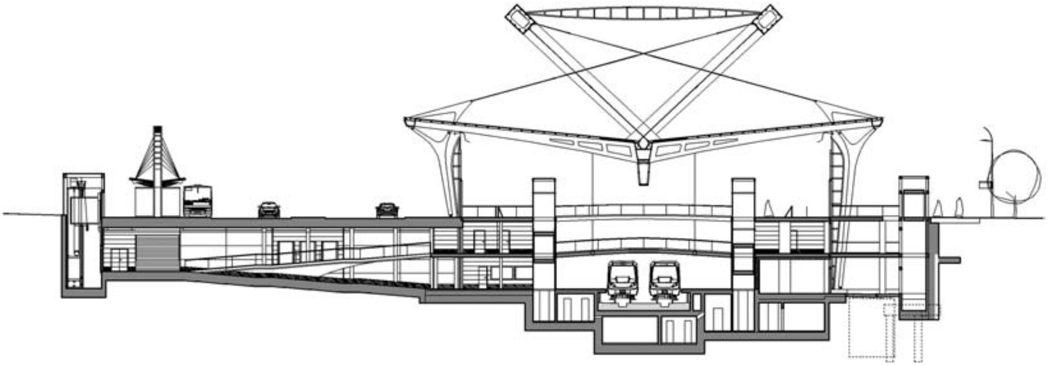

Figure 2. Cross section of the Střížkov station.

Figure 3. Design of the Střížkov station.

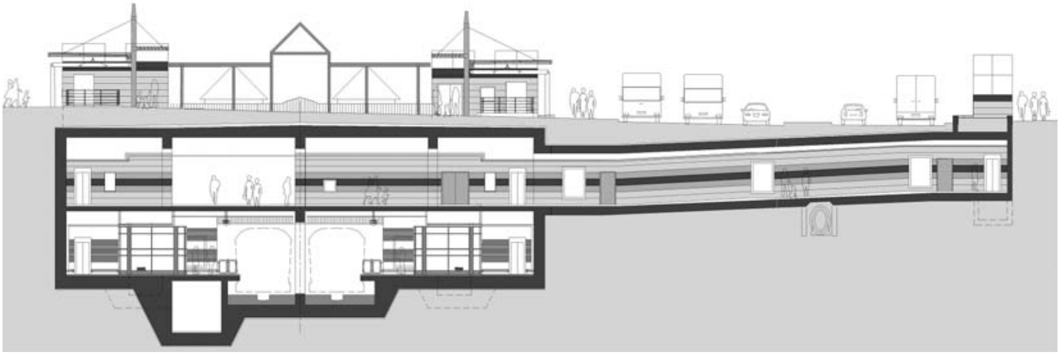

Figure 4. Cross section of the Prosek station.

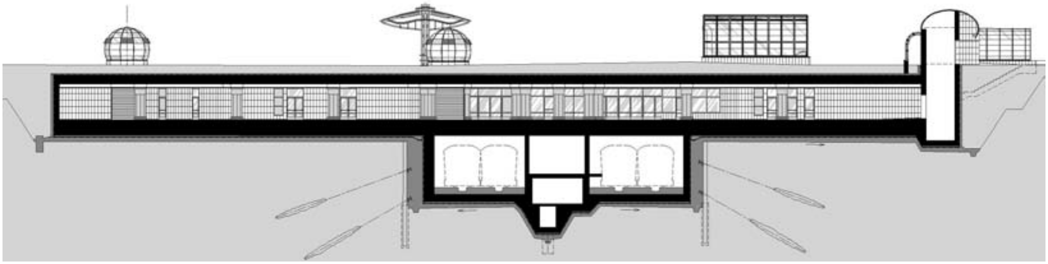

Figure 5. Cross section of the Letňany station.

skylights and exits with staircases and lifts for immobile people. The supporting structure of the station in the cross section consists of a closed cast-in-situ reinforced concrete frame with various internal divisions. The basic internal element of the internal division is a series of columns along the whole length of the station in the centre between the underground rails. The basic module interval of the columns in the longitudinal direction is 7.0 m. In terms of height, the structure of the station in the area of the platform and the technical-operating room is divided into three levels: the platform level, the level under the platform and the level of the vestibule with connected shopping passages and underpasses. In the remaining areas the division is into 2 height levels, i.e. the foundation slab under the underground rails and the intermediate ceiling at the level of the vestibule. A part of the underpasses with exits reaching beyond the ground plan of the station is naturally only at a height of one floor. The total length of the station structure is 210 m, the width ranges between 10–24 m. The height in the parts with three levels is 14.0 m and in parts with two internal levels it reaches 10 m.

3.3 *Letňany station*

The Letňany station is a long-term terminus located north of Kbelská St. and it should be a part of the entrance room of the future Prague exhibition ground replacing the existing temporary halls. The station is excavated with an island platform connected to 2 underground vestibules with exits ascending over the ground level. The platform level is 10.3 m under the landscaped surrounding ground.

The structure of the station in the cross section consists of a closed cast-in-situ reinforced concrete frame with an angular foundation slab in a constant width of 24.8 m in the whole station and a shunting track, i.e. a total length of 416.0 m. In the related part towards the Prosek station in a length of 176 m, the width of the structure continuously decreases to 16.4 m in the place of connection to the driven single-track tunnels. In the area of the platform the frame structure of the station is dipteral with a number of columns located in the axis of the platform in an interval of 6.4 m in the longitudinal direction. As regards the height, this part of the structure is divided by an intermediate ceiling at the level of the platform into the technological part under the platform with headroom of 2.3 m. The hall of the platform itself has a headroom of 8.35 m with a reinforced longitudinal girder over a number of columns and it will be covered by a lower ceiling lowered by 600 mm. Outside the platform space, the structure of the platform is prevailingly divided by longitudinal walls or wall pillars (in the shunting track) into a three-wing structure and in terms of height it is divided into three levels – the level under the platform, the level of the platform and the level of the vestibules. The total height of the closed frame of the station is 12.65 m, in the section of the shunting track and excavated tunnels in front of the station the structure is single-floored, height of 6.8 m.

4 TRACK TUNNELS

The design of the track tunnels related to the construction of the underground IV. C2 was based on the effort to construct two-track tunnels in the maximum possible extent. A great advantage of this design in the case of station tunnelled two-track tunnels consists in larger profiles of the tunnel, which makes it possible, especially if the New Austrian Tunnelling Method is used (NATM), to introduce high-performance station tunnelling techniques enabling flexible changing of the machines in the working face. This is reflected in increased speed of tunnelling, resulting in costs reduction of the tunnel construction.

A great benefit of this conception is mainly experienced in the excavated track tunnel between the Střížkov and Prosek stations. The pit excavated for the construction of the tunnel has a width of approx. 12 m. For the construction of 2 two-track tunnels connected to the station with island platforms, it would be 20.5 m i.e. 70% greater volume of excavation work. The aforementioned reasons had an essential impact on the design of the Střížkov and Prosek stations, which are designed with side platforms. On the contrary, as regards the long-term terminus in Letňany, the island platform is advisable for operating reasons.

Base inter-axial distance of the tracks in the cut two-track tunnel is 3.7 m and in the excavated two-track tunnel 3.6 m. This distance determines the diameter of the cross section of the proposed tunnels. In the sections aligning to the stations, the inter-axial distance of the tracks usually continually increases, and thus the cross section of the tunnels also increases in relation to the layouts of the station.

4.1 *Driven track tunnels*

A double-track tunnel was constructed here in the section of Ládví – Střížkov (975) m and in the section of Prosek – Letňany (1,280 m). The double-track tunnel was connected to two single-track driven tunnels in the direction of the Letňany station (length of 2×63 m). All cross sections of the double-track tunnels have a closed oval shape with stope surface of approx. 64 m^2. The tunnelling employed the NATM technologies – the tunnel was tunnelled by a primary lining of air-placed concrete with a thickness of 200–350 mm (based on the quality of the rock massif) reinforced by steel mesh combined with bar rock anchors. The tunnels were prevailingly tunnelled by horizontal division of the stope into a calotte and the lower part of the tunnel. In exceptional cases in a short section below the residential areas with thin hanging layer, the stope was also divided vertically. Following the completion of the tunnelling, waterproofing using a PVC foil, thickness of 3.0 mm, was installed on the front of the lining and the secondary lining was concreted using cast-in-situ reinforced concrete. This lining was made in 10-m strips so that the tunnel bottom was concreted at the first stage and the vault was concreted in a moving formwork at the second stage. The minimum clear width of the double-track tunnel is 8.9 m, the minimum clear height is 5.5 m and the minimum thickness of the internal steel-concrete lining is 400 mm.

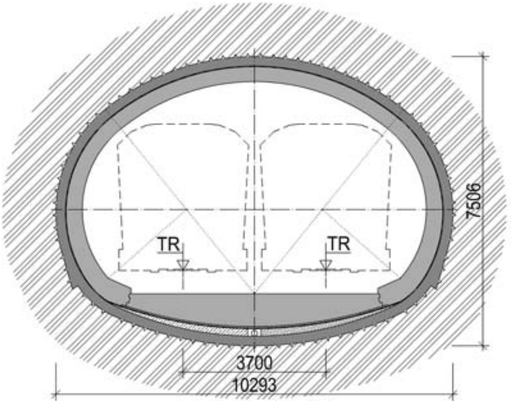

Figure 6. Cross section of the double-track driven tunnel.

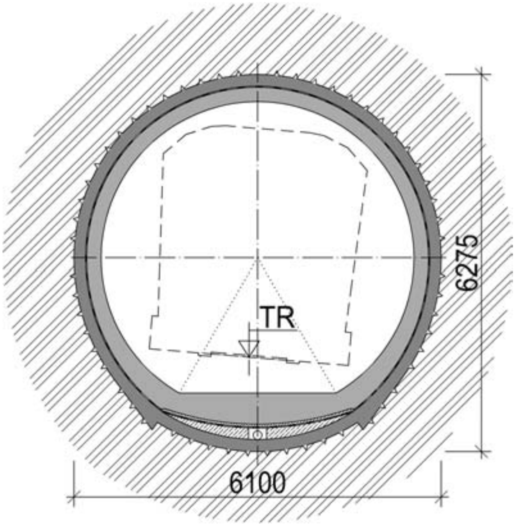

Figure 7. Cross section of the single-track driven tunnel.

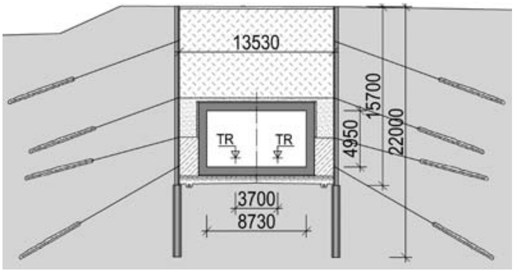

Figure 8. Cross section of the double-track excavated tunnel.

4.2 *Excavated track tunnels*

In construction of metro IV.C2 the two-track excavated tunnels are implemented of total length of 1251 m, of which in the Ládví – Střížkov section of 337 m, in the Střížkov – Prosek section of 772 m and in the Prosek – Letňany section of 142 m. Single-track excavated tunnels are not intended for the track. The double-track excavated tunnel has a minimum clear width of 8.4 m and a clear height of 4.95 m. From the structural point the simple closed frames of cast-in-situ reinforced concrete are designed. The thickness of the slabs and walls ranges from 700 to 800 mm, according to respective hydrogeological conditions. The tunnels were made in 12 m strips. On the inside, the tunnel is fitted with closed waterproofing PVC foil, min. thickness 2 mm with underlayers and protective layers. The tunnels were made in open excavation pit, secured with anchored strutted sheeting or pile timbering. The depth of the excavation pit ranges from 9.0 to 17.0 m. The backfill is performed to reach the original ground level above the tunnel roof.

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Planning for Canada's first bored road tunnel in over 40 years Kicking Horse Canyon Project

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ABSTRACT: The Kicking Horse Canyon Project (KHCP) involves upgrading approximately 26 kilometers of the Trans-Canada Highway in BC from the town of Golden to the western boundary of Yoho National Park to a modern four-lane standard. The KHCP is being undertaken in three Phases. Phase 1 comprised the replacement of the Yoho Bridge which was completed in 2004. Phase 2 comprises the replacement of the Park Bridge and grading work that is currently under construction, and Phase 3 comprises upgrading of the remaining stretches from the Yoho Bridge to Golden and the Brake Check to the Yoho Park Boundary. The preliminary design for the Phase 3 West section between Yoho Bridge and Golden is currently in progress and is evaluating new highway alignments through the steepest terrain section of the canyon that include two to three short (<1 km) tunnels as well as one alignment that includes a nearly 2.9 km tunnel that would be the longest bored road tunnel in North America. If constructed, the tunnel requirements for the highway improvement will represent the first major road tunnels in Canada in over 40 years. The preliminary design for the road tunnels comprises twin tube, 2-lane road tunnels. The road tunnels would be constructed within mixed sedimentary bedrock comprising limestones, dolostones, and shales with rock strengths varying from 30 MPa to 240 MPa. State-of-the-art road tunnel fire, life and safety requirements following NFPA requirements have been adopted given the increasing awareness for road tunnel safety measures. Comprehensive geotechnical site investigations have been completed as part of preliminary design studies. Portal location and tunnel design and constructability issues have been evaluated as part of the preliminary design work and are presented.

1 INTRODUCTION

The Kicking Horse Canyon Project (KHCP) involves upgrading approximately 26 kilometers of the Trans-Canada Highway in B.C. from the town of Golden to the western boundary of Yoho National Park to a modern four-lane standard. Due to the rugged nature of the canyon section and the requirements of the BC Ministry of Transportation this upgrading has required the consideration of tunnels as part of the preliminary stage of the project.

The KHCP is being undertaken in three phases. Phase 1 comprised the replacement of the Yoho Bridge which was completed in 2004. Phase 2 comprises the replacement of the Park Bridge and a major through cut in rock to a depth of 80 m that is currently under construction and planned for completion in late 2007, and Phase 3 comprises upgrading of the remaining stretch westwards from the Yoho Bridge to Golden and east of Phase 2.

The preliminary design for the Phase 3 section has evaluated several new highway alignments through the canyon that include two to three short (<1 km) tunnels as well as one alignment that includes a nearly 2.9 km tunnel. The construction for the Phase 3 section may commence in 2008/2009 and both traditional Design-Bid-Build and Design-Build-Finance-Operate (DBFO) contract approaches are being considered.

Both contract approaches have been implemented to date on past and current works. If constructed, the tunnel requirements for the highway upgrading represent the first major road tunnels in Canada in over 40 years since the construction of tunnels along the Trans-Canada Highway in the Fraser River Canyon. Most notably, the nearly 2.9 km tunnel option would be the longest road tunnel in North America.

Figure 1. Kicking Horse Canyon circa 1920's.

Figure 2. Kicking Horse Canyon.

Figure 1 shows a historical photograph of the highway through the canyon section.

2 PROJECT LOCATION

The Kicking Horse Canyon Project is located immediately east of the town of Golden, B.C. which is over 700 km northeast of Vancouver, B.C. and over 250 km west of Calgary, Alberta within the East Kootenay region of B.C. The section of the highway improvement project comprises 26 km of existing 2-lane highway from the town of Golden to the west gate of Yoho National Park. The Kicking Horse Canyon comprises a fairly narrow and winding canyon with maximum relief of nearly 500 m exhibiting very steep rock slopes as shown in Figure 2.

The elevation of canyon section of the alignment is approximately 1000 m and the main canyon section of the project is generally trending east-west with the Kicking Horse River flowing westwards through the canyon. The main CP Rail line is present along the lower slopes of the canyon and mostly along the northern side of the canyon below the existing

Figure 3. Project location and works phases.

highway. Figure 2 illustrates the project location and Phases of the project.

3 ROAD TUNNEL ALIGNMENT OPTIONS

Several studies have been completed since the early 1990's to identify highway improvement solutions for the Kicking Horse Canyon Project. The work from some of these studies identified several highway alignments along both the north and south sides through the canyon section. Recent preliminary work including terrain and natural hazard studies have indicated that a major ancient landslide is present along the south side of the canyon thereby precluding a highway alignment on the south side.

The remaining alignments along the north side of the canyon were evaluated further in terms of natural hazards and constructability in terms of maintaining the existing highway during construction. Following further work to date there exist two preferred alignment options referred to as the NB-2 and NC-2 alignments as shown in Figure 4.

The NB-2 alignment comprises a single, approximately 2.9 km tunnel that deviates northwards below a sharp ridge known as Frenchman's Ridge, passes under Dart Creek valley, a 400 m wide U-shaped hanging glacial valley, and continues eastward below the Black Wall Bluffs. The west portal is sited near the grade of the existing highway and the east portal is sited about 20 m below grade and at a sharp bend of the existing highway. Figure 3 shows the NB-2 alignment. The maximum cover along the NB-2 alignment is about 300 m below the Black Wall Bluffs. The minimum cover along the NB-2 alignment occurs within the Dart Creek valley and is under investigation at the time of writing.

The NC-2 alignment comprises three, short tunnels with lengths of about 555 m, 565 m and 565 m respectively for a total tunnel length of about 1685 m.

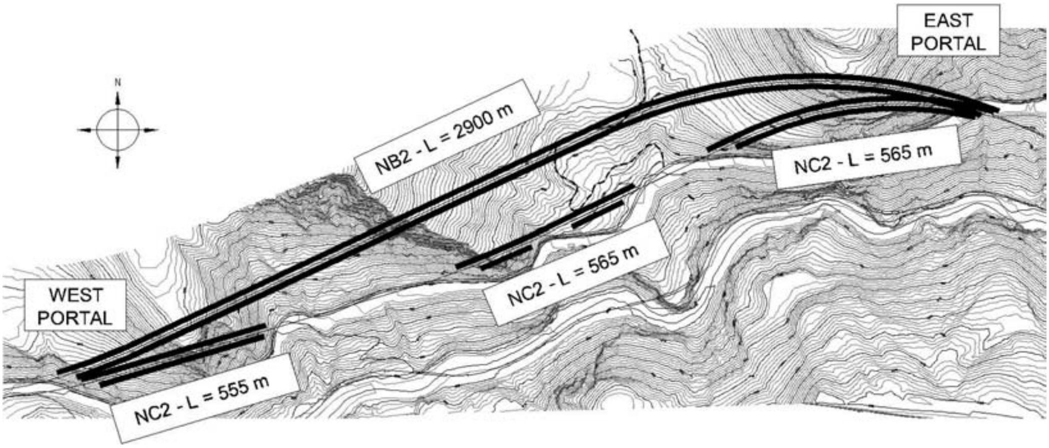

Figure 4. Road tunnel alignments.

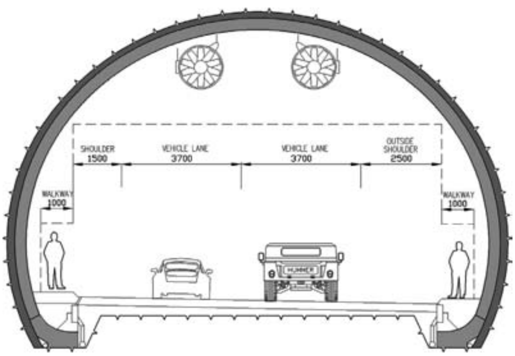

Figure 5. Large tunnel cross section.

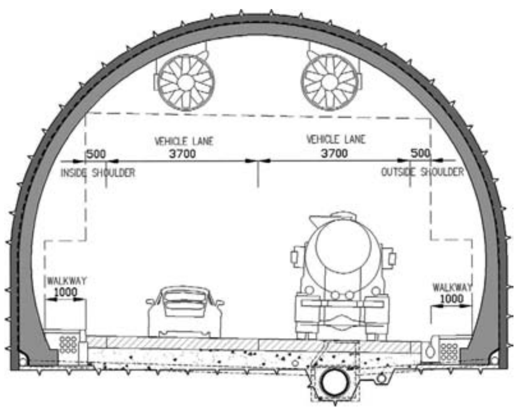

Figure 6. Small tunnel cross section.

The NC-2 alignment deviates not as much northwards as the NB-2 alignment but rather cuts through the series of bluffs/ridges along the canyon. The six portals required for the NC-2 alignment are sited both above and below grade of the existing highway.

With the development of possible new alignments through the canyon section it has been recognized that it is necessary to address the requirements of the external stakeholders. These requirements include maintaining access for CP Rail and river rafting businesses operating from upstream of the canyon, and to allow safe passage through the canyon for bi-cyclists. These requirements indicate that at least one lane of the existing highway will have to be maintained.

4 ROAD TUNNEL CROSS SECTION

Tunnel cross-section geometry was evaluated during the early stages of the preliminary design in recognition of the potential cost sensitivity and traffic safety requirements. As it was recognized that no tunnel roadway geometry criteria for road tunnels neither in Canada nor elsewhere in the world, a review of tunnel roadway geometry was undertaken and three options were developed for safety and costing consideration. The original tunnel cross sections varied from 115 m^2 to 127 m^2 and were based on lane widths of 3.7 m, walkways on both sides of 1.0 m, and varying shoulder widths of 1.5 m, 1.75 m, and 2.5 m, the largest allowing for emergency vehicle access (Figure 5). The initial tunnel cross section options were based on adopting large sized tunnels rather than minimized size tunnels similar to those in Europe. Cost comparison of these initial cross sections did not indicate significant cost increases for the largest size tunnel of 127 m^2. Smaller tunnel cross section options (90 m^2) have also been developed based on minimum shoulder dimensions for consideration based on acceptable international practice (Figure 6).

Figure 7. Geotechnical drilling by helicopter.

Figure 8. Existing rock cut.

5 GEOTECHNICAL INVESTIGATIONS

Comprehensive geotechnical site investigations have been completed as part of the preliminary design work for both tunnel alignments. Historical geotechnical investigations have been completed throughout the canyon dating as far back as 1985. The recent fieldwork has comprised seismic refraction surveys at all portal locations as well as across the Dart Creek valley, over 1900 m of rotary core drilling, point load strength testing, in situ packer permeability testing, and laboratory rock strength and abrasivity testing. Piezometers have been installed in many of the boreholes completed along the tunnel alignments.

Access to some of the boreholes at portal locations and along the tunnel alignments was difficult and was only accomplished by helicopter. All boreholes were drilled vertically and the deepest borehole completed was about 250 m along the Black Wall Bluffs. Figure 7 illustrates drilling of one of the deepest boreholes from the upper reaches of the Black Wall Bluffs by helicopter assistance.

6 TUNNELLING CONDITIONS

6.1 *Site geology*

The geology along the proposed tunnel alignments comprises sedimentary bedrock of the McKay Group and the lower part of the Glenogle Formation that has been subjected to eastward thrust faulting and folding with overturning. The McKay Group of rocks comprises five main sub-units (COMk2 to COMk6) that can be characterized as shaley limestones (odd numbered sub-units) and dolomitized limestones (even numbered sub-units). These five sub-units are present along the western and central sections of the alignment. The shaley limestones are finely laminated whereas the dolomitized limestones appear to be more massive in nature. The lower part of the Glenogle Formation can be described as a mixed slate/shale/dolomitic siltstone with finely laminated dolomitic beds.

Extensive bedrock outcrops are present in massive sub-vertical rock cuts formed by the original highway construction and appear along almost the entire tunnel alignments. Figure 8 shows a typical large rock cut of bedded limestone along the central section of the tunnel alignments.

6.2 *Rock strength and abrasivity*

Numerous point load strength index tests and uniaxial compressive strength (UCS) test were completed on the drill core. The rock strength of the shaley and dolomitized limestones generally varies from 15 MPa to 100 MPa with an average strength of about 60 MPa. The rock strength of the dolomitic shales and siltstones generally varies from 20 MPa to 240 MPa with an average strength of about 100 MPa. Figures 9 and 10 illustrate the variation of rock strength based on UCS lab testing results of the limestones and shales respectively. Rock abrasivity was evaluated based on CERCHAR Testing. CERCHAR Abrasivity Index (CAI) values for the limestones indicated values generally ranging from 0.8 to 2.4 characterizing the rock as slightly to non-abrasive. CAI values for the shales indicated values generally ranging from 2.0 to 4.0 characterizing the rock as very abrasive.

6.3 *Rock fracturing*

Rock fractures are pervasive within the limestone and siltstone/shale bedrock in the form of bedding and sub-vertical fractures that can be identified from both

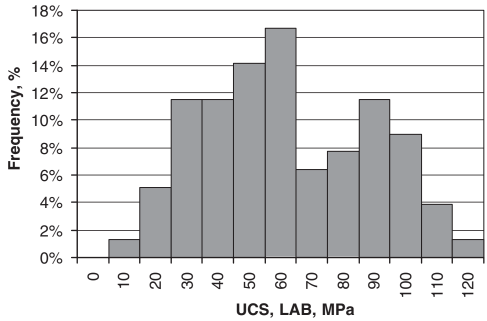

Figure 9. Rock strength – limestones.

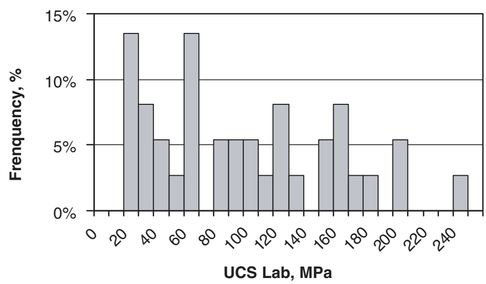

Figure 10. Rock strength – shales.

drillcore and surface outcrops. The dip of the bedding within both the limestones and shales typically ranges from 15 to 20 degrees. The dip direction of the bedding is northeasterly for the limestone and northerly for the shales. The main sub-vertical fractures sets are prominent as orthogonal fractures and are generally oriented both perpendicular and sub-parallel to the strike of the bedding. Bedding fractures are typically very smooth and planar. Figures 11 and 12 illustrate stereonets of the rock fractures within the limestones and shales respectively.

6.4 *Major fault/shear zones*

The geology along the tunnel alignments has been intruded by both sub-vertical and sub-horizontal thrust type faults. Sub-vertical fault/shears have been mapped from outcrops below the tunnel alignment along the CP Rail right-of-way and are expected to be present across the tunnel alignments. The most distinct sub-vertical faults are referred to as the West, Middle and Dart Creek Faults. The thicknesses of the West and Middle Faults are inferred to be no more than 10 m while the Dart Creek Fault is inferred to be as much as 20 m in thickness.

The most prominent major fault/shear feature along the tunnel alignments is referred to as the Black Wall Bluffs Fault. This fault zone is a thrust fault that is clearly visible in a sub-vertical rock cut at the western edge of the Black Wall Bluffs. This zone is comprised of highly distorted siltstone and shale bedrock as shown in Figure 13 and is currently inferred to be undulating in nature and may also be present near the east portal. The overall orientation of this fault is inferred to be dipping less than 20 degrees towards the northeast. The fault is present in the exposed rock cut over a vertical height of about 15 m and may have an overall thickness as much as 25 m based on a limited number of borehole intersections from targeted drilling. It is noted that complete recovery of this fault was obtained in one of the boreholes that were targeted.

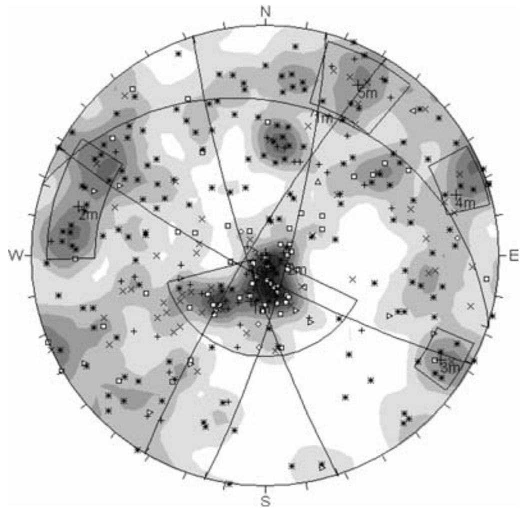

Figure 11. Rock fractures – limestones.

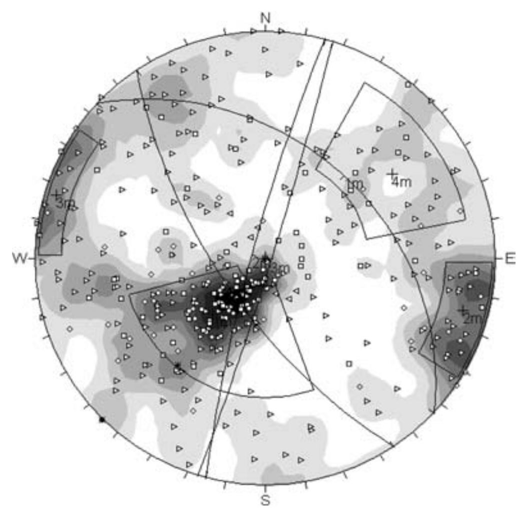

Figure 12. Rock fractures – shales.

Figure 13. Black Wall Bluffs Fault exposure.

Interpretation of the possible length of intersection of this major fault with the proposed tunnels has been made based on a 3D geological model using surface outcrop and borehole data. Results from the model suggest that the length of the intersection of this fault with the tunnel could be as much as 250 m extending over much of the eastern portion of the tunnel alignment. The geotechnical relevance of this fault zone in terms of construction cost is not considered to be significant enough to justify modification of the tunnel alignment to reduce this intersection length versus the possible increased costs associated with higher capacity tunnel support that may or may not be necessary.

6.5 *Groundwater conditions*

The groundwater table is inferred to be at a shallow depth below surface along the tunnel alignments. A limited number of packer permeability tests were completed in the dolomitized limestones along the Dart Creek valley section of the tunnl alignments. These tests indicated rock mass permeability of this generally massive bedded bedrock is in the order of 5×10^{-7} m/s to 5×10^{-10} m/s. Given the relatively low cover along the alignments, the magnitude of groundwater inflows from most of the bedrock is not expected to be significant. Significant groundwater inflows can however be expected from the identified sub-vertical faults at Dart Creek valley and possibly from the Black Wall Bluffs Fault.

Artesian groundwater conditions are indicated from two boreholes completed into bedrock in Dart Creek. These conditions are inferred to represent groundwater flow under pressure either along the top of bedrock in the Dart Creek valley or within the extensive thickness of glacial overburden in which the Dart Creek stream course has disappeared and then re-appears at a location on the north side of the existing highway. These artesian groundwater conditions may pose problems for surface excavation in the overburden at Dart Creek but are not expected to have any influence on tunnel excavation in bedrock below Dart Creek valley.

7 FIRE, LIFE AND SAFETY DESIGN ISSUES

The fire, life and safety design requirements for road tunnels is currently undergoing significant changes and improvements as a result of the recent series of significant fires within major road tunnels in Europe. No such requirements are established for road tunnels in Canada as a code of practice and therefore the current approach for the proposed tunnel options is to adopt those standards set out by the National Fire Protection Agency (NFPA) of the United States that are considered to represent state-of-the-art industry practice. These standards set out requirements for the maximum spacing of pedestrian cross passages, fire suppression, communications, lighting, ventilation, and operations monitoring. In Europe, there currently exist a number of research groups dedicated to establishing a suggested code of practice for fire, life and safety requirements for road tunnels. The cost of the fire, life, and safety requirements for road tunnels is a significant portion of the overall construction costs. Significant costs are also associated with providing a tunnel operations center and operations monitoring and maintenance.

8 CONSTRUCTABILITY ISSUES

8.1 *Tunnel excavation and support*

The stability of the tunnels will be predominantly influenced by the formation of potentially unstable rock wedges formed along the crown and haunches due to the presence of the pervasive bedding in conjunction with orthogonally oriented sub-vertical fractures. Large bedding slabs can be expected to form and be unstable, requiring regular support for stability and safety.

Owing to the generally strong and bedded nature of most of the bedrock it is expected that the tunnels can be excavated by a standard 2-stage top heading and bench approach or even possibly full face method depending on the final size of the tunnels. Conventional tunnel support measures comprising pattern rock bolts with mesh and shotcrete are expected to provide adequate support for the majority of the tunnels. Enhanced tunnel support measures comprising lattice girders in conjunction with mesh and shotcrete and possibly supplemented with forepoling or self-boring anchors may be necessary at the intersection of major fault zones such as the Black Wall Bluffs Fault.

Kinematic and numerical analyses have been competed to assess excavation stability in terms of the

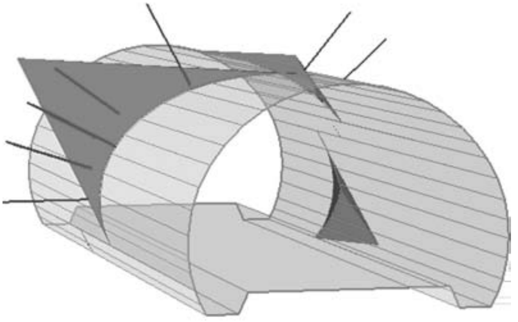

Figure 14. Typical large unstable wedge.

maximum size of potentially unstable wedges formed around the tunnels and any potential for overstressing to confirm the adequacy of conventional tunnel support requirements. Figure 14 shows an example of a large unstable wedge that may form along the sidewall of the proposed tunnels that would be required to be supported using pattern rock bolts during excavation.

8.2 *Portal excavation*

The proposed portal locations are sited in very close proximity to the existing highway. Bedrock is present at very shallow depth for most of the portals for the NC-2 alignment and the west portal for the NB-2 alignment. The main challenge for excavation of the portals in rock will be controlled blasting and management of traffic due to the close proximity of the existing highway.

In comparison, mixed overburden materials are present to a depth greater than 10 m at the east portal of both alignments. The nature of these materials is not well defined but is believed to comprise loose side-cast material from the construction of the original highway.

The east portal is sited below the existing highway and presents a challenge for excavation and support in order to prevent any impact to the existing highway. An initial portal layout was developed based on commencing tunnel excavation in bedrock for minimum tunnel costs that requires significant excavation into the existing highway as shown in Figure 15.

A preferred alternative to this layout has been developed based on commencing tunnel excavation as soon as possible with minimum cover through overburden material as shown in Figure 16.

This alternative layout provides an appropriate buffer from the existing highway however would require specialized tunnel support measures including some form of pre-support such as forepoling or self-boring anchors at a higher cost for the initial section of tunnel.

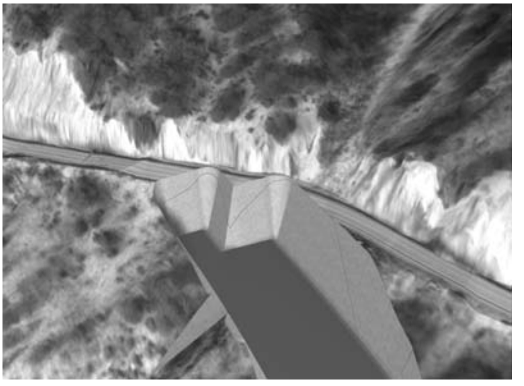

Figure 15. East portal excavation in rock.

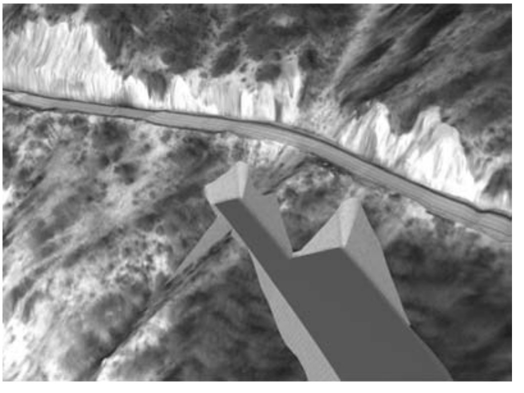

Figure 16. East portal excavation in overburden.

A portal excavation layout for the east portal that does not impact the existing highway also provides the benefit that the highway serves as an avalanche and rockfall catchment bench during operations. The east portal is located immediately adjacent to hazardous avalanche chutes that would otherwise require some form of protection-shed structure at the east portal.

8.3 *Dart creek intermediate access tunnel*

Owing to the expected relatively large portal excavations required to be formed prior to tunnel excavation, it has been recognized that the low cover section of the Dart Creek valley offers an opportunity for the construction of an intermediate access tunnel and laydown area that provides great benefits to the project. In addition to independent access for tunnel excavation, the intermediate access tunnel prevents any need for muck haulage along the existing highway to the designated spoil disposal site in the Dart Creek valley. Also, the intermediate access tunnel may be used for permanent emergency access during operations from

Figure 17. Dart Creek intermediate access tunnel/laydown/spoil site.

a tunnel control center that is currently proposed to be located near the access tunnel at Dart Creek.

The proposed intermediate access tunnel would be constructed entirely in bedrock after surface excavation and the establishment of an appropriate laydown area located immediately north of the highway in Dart Creek as shown in Figure 17.

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Preparation of metro's a line extension in Prague

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ABSTRACT: Metro network is the backbone of the public transport system of any large city and, from this point of view, a further development of this system is a substantial question and one of the priorities. The current Metro network in Prague consists of three lines: A, B and C, with 53 stations and the total line length of approximately 54 km. There is a dedicated depot for each line (in Hostivař, Zličín and Kačerov). One of the primarily monitored construction projects in the Metro network is currently the extension of Metro's A line westwards. Given the overall extent of the project, it is envisaged that the project will be divided into stages: – The operation section of the A line, from Dejvická to Motol; the newly considered extension westwards to the area of the Petřiny housing estate and to the Motol Teaching Hospital; operation length of 6.0 km, 4 stations. If required, this operation section can be flexibly shortened so that it will terminate at Petřiny. (operation length of 4.5 km, 3 stations), – The continuing operation section Motol – Prague Ruzyně Airport (the north branch), operation length of approximately 7 km, 4 stations. A study work is currently in progress to check the construction and technical feasibility of the route designed in this way (the penetration through the Prague Ruzyně Airport in the lane of railway transport, new station layout, etc.). The implementation of this route, if any, directly depends on how it will be actually needed in relation to the transport from and to Prague Ruzyně by rail, – South branch – Motol – Zličín, operation length of 5.0 km, 3 stations. The Motol – Bílá hora section is identical with the route of the above-mentioned north branch of the line, further continuing to the Řepy housing estate and the Zličín station (approximately 3.8 km). In this regard, the line can be branched at Bílá hora. This south branch is only viewed as a prospective development of the Metro network in this quadrant of the city.

1 INTRODUCTION

An extension of the Metro Line A is planned to generally improve the quality of the City's northwest sector traffic service by capacitive rail traffic. The improvement should become evident by a total reduction of bus lines in this area of Prague. In combination with the so far considered railway connection, a possibility to achieve a tranfer terminal in the area of Dlouhá Míle and Ruzyně Airport via a new track has been considered, having no impact on the railway fixed under the Prague Capital's City Plan. At the same time, a plan to remove the existing Metro Line A terminus from the Dejvická station to a more suitable and less exposed location, in terms of town planning, towards the outskirts of the Town, has been considered.

2 GENERAL DESIGN PRINCIPLES

Within the context to the new facts, the planned solution represents a basic change of the concept of the Metro Line leading to the area of Dědina housing estate, then actually directly to the Dlouhá Míle terminal and, finally, to the Ruzyně Airport alone. At the same time, an effort has been maintained to keep the advantages of the initial solution, enabling the basic route to branche in the area of Bílá Hora to Řepy housing estate and Zličín terminus. It is possible to consider the priority of the selected direction of routes' leading according to specific needs in given time. In light of progress of the line construction, the possibility to terminate its phase in the Petřiny station has been moved to the Motol station.

An expected final option of the solution is to lead the line from the existing Dejvická station in the track Červený Vrch – Veleslavín – Petřiny – Motol – Bílá Hora – Dědina – Dlouhá Míle – Ruzyně Airport with a possibility of branching behind the Bílá Hora station to Řepy – Zličín direction with a linkage to Zličín Depot of the Line B. A subject matter to modify the line via Ruzyně Old Airport with a possibility to insert another station into such area was verified as a consequence of reaction on Metro routing in a new track led through Prague Airport area of interest.

In terms of train traffic operation, railcards for train set turning and deposition are dislocated en route within the context of the particular stations, taking

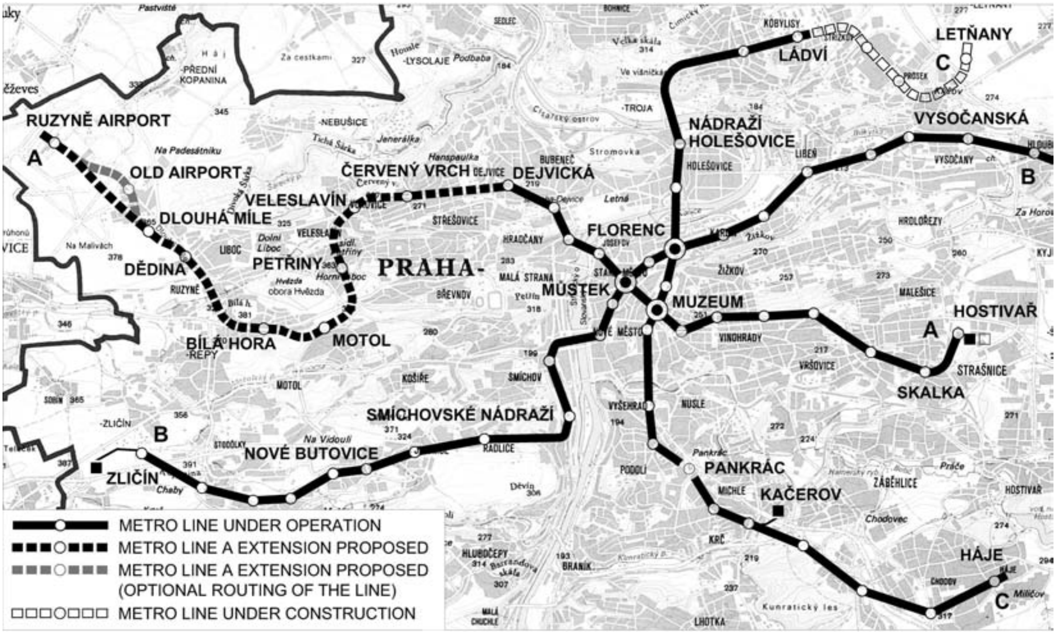

Figure 1. Section of the Prague Metro network.

into account a possibility to put the Line into service step by step as well as traffic organization during extraordinary operation situations and variability of service train rides to provide the maintenance during regular night exclusions. The railcard for set turning and deposition is projected in the Ruzyně Airport Terminal station as well as in the Motol and Bílá Hora Phases terminal stations. Such arrangement enables train turnings up to ca. 5 km down the Line.

According to standards, the following description of the Line's final solution was compiled down the stationing direction, i.e. from Ruzyně Airport to Dejvická stations. On the basic level of knowledge, it discusses the designed issue in terms of specific stations and sections between the stations.

2.1 *The Ruzyně Airport station*

The Ruzyně Airport station is a driven single-nave station with a central platform in depth of 20.9 m below ground surface. Its location, height position and links to the area comply with the Airport's development plans and expected railway terminus of the railway line Airport – Town Centre. At the same time metro terminus location accepts possibility of further extension of railway line under the new Sever II Airport Terminal's building and expand the stop by including another rail with a platform on the northern side.

The station is designed as terminus on a long-term basis, having a rail arrangement to enable turning and depositions of train sets. Therefore, siding and stabling rails in a four-rail arrangement are designed behind the station. The station tunnel including coherent the siding and stabling rails is regarded as a cavern with cross section of the excavation area of about 220 m^2 in length of 450 m. The cavern shall be driven by NRTM with a vertical and horizontal excavation segmentation of the stope. It will be partially driven in Jurassic rocks (calotte) and partially in Upper Palaeozoic and Proterozoic rocks. Two single-track tunnels 2310 m long will connect terminus with the station Dlouhá Míle. The tunnels will be largely driven in Upper Palaeozoic and Proterozoic rocks. They will rise to Jurassic formations solely in the neighbourhood of the Dlouhá Míle station. The tunnels will be performed by TBM with mounted reinforced lining. An open building pit for the Dědina station and adjacent tunnels will be used for deployment of TBM complex. Total length of the open building pit will be 450 m. The sequence of the works has to enable passing of TBM through the Dlouhá Míle station.

2.2 *The Dlouhá Míle station*

In terms of its location, the Dlouhá Míle station is fixed to have a direct linkage to the stop included in the Airport and Town Centre railway connection, similarly as the Ruzyně Airport station. The main importance of the station generally lies in an option to form a quality traffic terminal, especially for transfers from

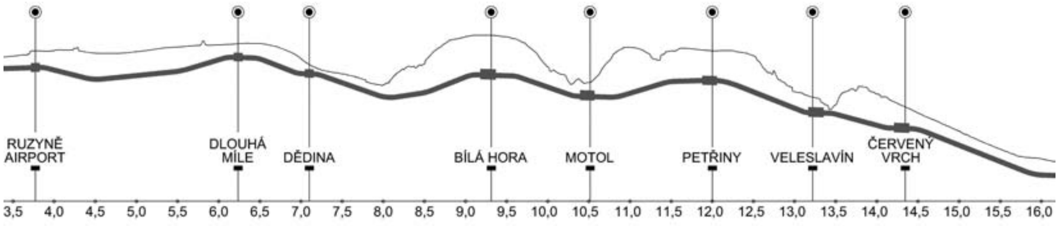

Figure 2. Exaggerated longitudinal profile metro line A extension.

long distance bus lines and regional bus transports. The junction also comprises a capacity parking (P + R System), located in a close linkage to the Metro and railway station on its eastern side.

The station is designed as driven station with a platform 17.3 m under the ground surface across under the projected railway stop. In this way a direct interconnection of the Metro station central platform to both railway stop departure edges is ensured, serving as departure edges of coherent bus transport as well. A direct exit from the Metro station to the surface via lift systems ended in the P + R parking premises is designed in the south part of the station.

The Dlouhá Míle station including the section of rail crossover behind the station is regarded as a cavern with a cross section of ca. 220 m^2. The excavation is regarded by use of NRTM with a vertical as well as horizontal segmentation, similarly as in the Ruzyně Airport. It will be driven in Jurassic formations. Two single-track tunnels 620 m long towards the Dědina station will follow the cavern. Again, TBM is considered concerning the tunnel excavation. As mentioned above, it is considered that the TBM will be deployed at the Dědina station. The track tunnels will be largely driven in Jurassic rocks, however, they will dip into Ordovic layer environment in front of station Dědina.

2.3 *The Old Airport station – an optional routing of the line*

The direction of the above described general routing of the Line is re-arranged in the section between Ruzyně Airport and Dlouhá Míle stations in order to be possible to insert another metro station near the Old Ruzyně Airport. Driven one nave station was proposed with a platform in depth of 12.2 m below ground surface. Distances between the station's centre and Ruzyně Airport/Dlouhá Míle stations make 1710 m, eventually 917. Total prolongation of the Line using this option then makes ca. 150 m.

The building pit premises of the driven Old Airport station would be utilized to deploy TBM to excavate single-track tunnels both in direction to the Ruzyně Airport and Dlouhá Míle stations and then to the Dědina station, where the pit may be utilize either to dismount the TBMs or turn and subsequent reverse excavation. In this option, the track tunnel will rise up to the environment of Jurassic formations ca. 500 m before the Old Airport station, in which they will be driven up to the Dlouhá Míle station.

2.4 *The Dědina station*

The Dědina station is designed as a driven one nave station with a central platform in approximated depth of 12.2 m below ground surface. It is located northwest from the Krnovská – Vlastina Streets crossing. The distance between the station's centre and a following The Bílá Hora station is 2211 m.

Beyond the Dědina station, the line sharply slopes down to Ruzyně area to safely undergo the existing residential area. Then it grades up to Bílá Hora, to the following station. The maximum longitudinal gradient is 39.5%. The line routes through Litovice Brook flood plain in this section. Saturated quaternary sediments in the area where the proposed Metro single-track tunnels undergo the bed of the brook have approximate thickness of 8 to 10 m. The metro vertical alignment is designed approximately 8 m under the base of the mantle formations. The bedrock here is formed by Ordovic slates of Letná strata. It is possible to excavate the designed single-track tunnels by TBMs in such rock environment and to warrant very small gradients of the existing Ruzyně low-floor residential area as well. The TBM will be deployed in the section of the open building pit beyond the Dědina station and will be dismounted either in the cavern of the driven one nave station Bílá Hora or as long as the station is passed in the area near to stabling rails before the Motol station.

2.5 *The Bílá Hora station*

The Bílá Hora station is situated transversally under Karlovarská Street between Karlovarská – K Motolu and Karlovarská – Turnova crossings. The station is driven with a platform in depth of 49.2 m below ground surface. One underground vestibule situated under the centre of the existing tramway line loop is considered. A thickness of the quaternary mantle formations moves in the range of 2 to 6 m. The rock overburden of the station is largely formed by Jurassic formation sediments. The stratum is about 35 m high. However, the station itself is situated to Ordovic strata. It is possible to project any type of driven station including the most sophisticated one nave station under such given geological conditions.

The line continues through driven single-track tunnels to the area near to the Motol Hospital's northern entrance, where a metro station of the same name is located. The section has a maximum longitudinal gradient of 34.6%, and is approximately 1100 m long.

2.6 *The Motol station*

The Motol station is located north of Kukulova Street in the service area near of the Motol Hospital area's northern entrance. The station is orientated in east-west direction with a single vestibule accessible from the west front of the platform. A subway under Kukulova Street follows the vestibule. The station is designed as shallow driven station with a platform in depth of 18.1 m below ground surface. Concerning the projected vertical and horizontal alignment of the track, the station is situated in very unfavourable geological conditions. A slumping gradient at the edge of the Jurassic plate is in the overburden of the station. The bedrock where the station would be executed is formed by Ordovic strata. It would be suitable to consider that the station should be driven, preferably consisting of two separate tunnels with a sufficient rock intermediate pillar, interconnected by transversal corridors followed by the access to the station.

Due to the expected phasing of the works the station is equipped as a temporary terminus with a rail yard for set turning as well as deposition during the phase. Three-rail arrangement beyond the station (against the direction of stationing) is expected. Within a context of standard requirements, the direction and height design of turning rails is different from the track rails of the future status. A rectification of the section will be necessary if the line will continue in future.

The line turns to the north via a left-oriented curve beyond the Motol station and is led to the Petřiny station. The driven single-track tunnels are 1085 m long.

2.7 *The Petřiny station*

The Petřiny station is situated under Brunclíkova Street with a centre approximately in a prolongation of Fajmanové Street. The station is designed as a driven one nave station with a platform in depth of 36.5 m below ground surface. It has a single underground vestivule located before a crossing with Na Petřinách Street. A subway with exits on both sides of the Na Petřinách Street including exits to rearranged tramway platforms follows the vestibule. Eventually, a long direct section and track tunnel gradient conditions in direction to the Motol station allow to execute the station as a temporary terminal as well (in frame of phasing of the works). Siding and stabling rails a four-rail arrangement are designed concerning the station. Due to operation and technological reasons, such arrangement is completed by two single rail junctions (before and beyond the station), allowing to achieve a maximum operation's versatility.

A thickness of mantle formations moves around 2 m. There is a bulky Jurassic formation strata of app. thickness 18 m and a bedrock, in which a cavern of the station including shunt rails will be driven, is formed by Ordovic rocks. A thickness of rock missive's overburden without the mantle formations will move around 26 m. The line turns to the northeast via a curve beyond the station and is led to the Veleslavín station. The section has a maximum longitudinal gradient of 39.5%, and the driven single-track tunnels are 1110 m long.

2.8 *The Veleslavín station*

The Veleslavín station with a centre approximately under Evropská Street is situated in the area between the existing ČD line (the Veleslavín Railway Station) and eastern section of K Červenému vrchu Street. The station is designed as shallowly driven with a platform in depth of 19.4 m below ground surface. It has a single underground vestibule situated between the ČD line and Evropská – Kladenská Streets crossing. The location of the station, vestibule and individual exits is designed to allow direct linkages to a temporary bus terminal during the phase and at the same time to respond in advance to the planned modernization of the existing ČD line without any need for reconstruction.

The projected routing direction and height of the line and given geological conditions allow designers to propose a construction of a two naves driven station with a row of bearing pillars in the axle of the station, on which a reinforced girder supporting inside footing of station tunnel vaults is placed. Such station's design allows a sectioning progress of the station's construction even in less convenient geological conditions and a low height of the overburden. To ease a dispersal of passengers from escalators to the area of the driven part of the station, the driven part situated under Evropská Street may be combined with the excavated section to the Petřiny station, where the exit escalators will be placed and the structure will have no internal supports.

In direction to the Červený Vrch station, excavated track tunnels will follow the driven part of the station between which a technological part of the station may be placed. The line then continues as driven up to the Červený Vrch station. There is a maximum gradient 26.9% in the section.

Table 1. Basic information on the line.

	Final status	Motol phase
Construction length of the projected section	12.793 7 km	5.663 8 km
Operational length of the projected section	12.7 km	6.0 km
Number of the stations	8	4

Table 2. Basic Information on the stations.

Station name	Station type	Platform depth under ground surface, m	Distance between stations, m
Ruzyně Airport	driven	20.9	
			2465
Dlouhá Míle	driven	17.3	
			866
Dědina	cut and cover	12.2	
			2211
Bílá Hora	driven	49.2	
			1202
Motol	driven	18.1	
			1485
Petřiny	driven	36.5	
			1220
Veleslavín	driven	19.4	
			1125
Červený Vrch	driven	26.7	
			2175
Dejvická	existing		

2.9 *The Červený Vrch station*

The Červený Vrch station is situated under Evropská Street, in the area between Evropská – Arabská and Evropská – Horoměřická Streets crossings. The station is designed as driven one nave station with a single excavated vestibule. The platform's depth under ground surface is 26.7 m in the centre of the station. The rock environment around the station is formed by Skalec quartzites with a thickness in the overburden of the station between 13 and 22 m.

Table 3. Basic information on line's optional routing.

	Final status	Motol phase
Construction length of the projected section	12.946 4 km	5.663 8 km
Operational length of the projected section	12.9 km	6.0 km
Number of the stations	9	4
Prolongation of optional route	153 m	

Table 4. Basic information on the stations (Line's Optional Routing)

Station name	Station type	Platform depth under ground surface, m	Distance between stations, m
Ruzyně Airport	driven	20.9	
			1701
Old Airport	cut and cover	12.2	
			917
Dlouhá Míle	driven	17.3	

The vestibule accessible from the eastern front of the station is situated to the crossing with Horoměřická Street with a linkage to considered small terminal of town and suburban bus lines, destined especially from Nebušice, Jenerálka and Horoměřice areas. A subway under Evropská Street follows the hall, allowing a direct transfer to the existing tramway line on Evropská Street as well.

Beyond the Červený Vrch station this Metro Line continues via two one-rail track tunnels 1835 long and ends in shunt rails of the existing Dejvická terminus station on the Prague's Metro Line A. The track tunnels decline in a maximum gradient of 39.5% in direction to the Dejvická station. They will be largely mined in quaternary soils and below the underground water level. A tunnel construction by use of excavating machines is considered with regard to geological conditions and considerably big length of the Line's section.

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Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina & Romancov (eds)
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Preparation of Metro's new D Line in Prague

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ABSTRACT: Metro network is the backbone of the public transport system of any large city and, from this point of view, a further development of this system is a substantial question and one of the priorities. The current Metro network in Prague consists of three lines: A, B and C, with 53 stations and the total line length of approximately 54 km. There is a dedicated depot for each line (in Hostivař, Zličín and Kačerov). One of the primarily monitored construction projects in the Metro network is currently the construction of the new fourth Metro line, line D. The operation length of its first planned operation section is 10.2 km with 10 stations. Given the overall extent of the project, it is envisaged that the project will be internally divided into two or three operation sections, which will directly follow one another. The implementation of Metro's new D line has been monitored for a long time and gradually stabilized into the current form. The D line, located in the southern sector of the city, is a completely new rail transport element, thus reacting to the current completely inconvenient situation when buses are used as the means of public transport there. What is more, the buses mostly transport the passengers to Metro's C line (the Kačerov station) and partly also to the B line (the Smíchovské nádraží station). The transport characteristics of the D line are different from those of the currently operated Metro lines because surveys show that the D line traffic will be much lower than that of the other Metro lines. The use of a new light system is being considered for the Prague Metro, its new D line, in view of the above-mentioned facts and the worldwide trends of the overall modernization of rail transport systems. The 'light Metro' is viewed as the base of a new transport system, which will serve the city and can also penetrate into the region.

1 INTRODUCTION

The Line D is the fourth diameter in the Prague metro network. Its objective is to provide radial transit to the centre from the areas of Krč, Lhotka, Libuš, Nové Dvory, Písnice and the development territories on the southern edge of Prague.

The first operational section of the Line D should partially improve the operational situation on the Line C. By opening the Line D to traffic, a portion of the traffic load will be transferred from the Line C to the Line D, thus the train occupancy on the operating Line C will be partially reduced.

The main benefit of the first operational section of Line D must, however, be sought for in the replacement of the extensive bus traffic leading along Vídeňská Street, and a decrease in the high passenger traffic intensity at the Kačerov metro station. At the same time we can expect that the passenger interchange volume at the Muzeum station will also be diminished.

Not negligible advantage is also elimination of operational vulnerability of the Line C on the Nusle Bridge, as well as the development of a new metro line from the southern region of the city to the city centre. The pre-stressed reinforced concrete structure of the Nusle Bridge is threatened in the long term by the effects of stray currents, and by the fact that the need for overall reconstruction of the bridge in the future cannot be excluded.

A section of the metro network with the three operating lines A, B and C and the Line I D under preparation, is shown in Fig. 1. The Line C is shown including its section IV C2 Ládví – Letňany, which is under construction with the completion scheduled for the middle of next year.

2 BASIC DATA

- Region: the City of Prague
- Employer: Municipality of Prague
- User: Dopravní podnik hlavního města Prahy (a passenger transport authority)
- Construction period: commencement after 2008

3 ROUTE DESCRIPTION

The basic range, or the sphere of service provided by this section of the Line D marked as I D is currently understood to cover an area from Písnice to the Hlavní

Nádraží station. Variants to the solution are focused specially on the vehicle category and corrections in the details of the horizontal and, above all, vertical alignment associated with the selected vehicle category. At the same time, the issue of compatibility with the other kinds of rail transit existing in the PID (Prague integrated transit system), is under consideration, from the aspect of possible utilization of the system in a regional scale.

The above-mentioned extent of the line differs from the expectations of the planning department of the City of Prague. It roughly represents unification of the originally discussed two operational sections of this line (the section I D from Náměstí Míru to Nové Dvory and section II D from Nové Dvory to Písnice), with an extension comprising one interstation section toward the south (a part of the originally discussed section III D) to the Hlavní Nádraží metro station. The internal division of the project and of both the process of opening the Line I D to traffic into phases has not been stabilized unambiguously; it will be a subject of further design preparation.

Figure 1. Section of the Prague Metro network.

A total of 10 metro stations have been included into the design of the first operational section of the Line D, i.e. the Hlavní Nádraží, Náměstí Míru, (the crossing with the Line A), Náměstí Bratří Synků, Pankrác, (the crossing with the Line C), Olbrachtova, Nádraží Krč, Nemocnice Krč, Nové Dvory, Libuš and Písnice stations.

It will be possible to design 60–100 m long trains for the Line D, considering the transport demand. The line will be serviced by a new depot in Písnice with a planned capacity of 20 trains 100 m long.

The newest documentation prepared by METROPROJEKT Praha a.s. in 2004 in two variants agrees with the positions of stations and general horizontal alignment of this line determined by previous studies.

The dissected topography of Prague, the considerable differences in elevations of individual valleys and terraces above them (e.g. the Vltava River valley, Nusle valley, Pankrác terrace, Krč valley, Nové Dvory and Libuš terraces, etc.) puts heavy demands on the design of the alignment for lines using traditional rail systems. Within the scope of rail systems used for urban mass, traditional heavy metro rail lines with their standard design parameters (horizontal curve diameter $R \geq 500$ m, rising gradient up to 40‰, stations on straight line or up to a radius $R \geq 800$ m) are especially difficult to accommodate to Prague terrain configuration. This often results in stations placed at great depths, with demanding access roads, time losses for passengers, and significant operational costs, incurred namely due to escalators (see Fig. 2).

With respect to the above problems and because of the fact that the order had specified neither a particular system nor a vehicle, the following basic design parameters of a light rail mass transit system were formulated at the beginning of the work on the design:

Horizontal curves on a running track: in general $R \geq 305$ m (80 km/hour)

Horizontal curves at a station: min $R = 300$ m (at max. platform length of 100 m)

Maximum rising gradient on a running track: 60‰

Maximum rising gradient at a station: 15‰

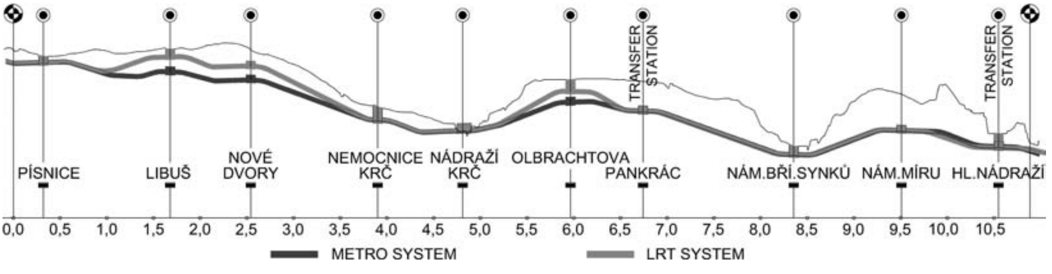

Figure 2. Exaggerated longitudinal profile metro line I.D of the Variant 1 and 2.

3.1 *The variant 1*

The variant 1 represents the conception of the Line D as a “traditional metro” in the meaning of the principles developed by the planning department of the City of Prague, excepting the overall extent of the Line I D and the route deviation from the originally intended Zálesí station to the Nemocnice Krč station. The operational length of this variant amounts to 10.2 km.

3.2 *The variant 2*

The variant 2 deals with a light rail mass transit concept. Regarding the horizontal alignment, it keeps the horizontal alignment designed for the traditional metro (Variant 1), but individual stations are closer to the surface, as allowed by local conditions. The Zálesí station is, similarly as in the variant 1, replaced by the Nemocnice Krč station. The Hlavní Nádraží station is situated in a position set off in the northwest direction to the close vicinity of railway areas. The operational length of this variant of the route is also of 10.2 km.

It must be added for the sake of completeness that also legislative issues will probably have to be solved should the light rail system or another similar system be introduced. The technical level of today produced trains eliminates differences between metro cars and tramway cars. The LRT or light metro conception has been used for rather a long time. Those trains are capable of safe operation in the conditions of tramlines while providing travelling comfort of traditional metro. No explicit regulations exist currently for this track conception. The main issues are the spatial arrangement of railway structures and geometrical configuration of rails; the current regulations valid for tramlines allow more economic alignment, differing clearance profiles for the tramline and metro, and other aspects. The Ministry of Transport, the Department of Railway Tracks and Railway Transport, are not preparing any regulation for the light metro or LRT. It would certainly by possible to start from regulations used in other EU member countries.

4 DESCRIPTION OF THE VARIANT 1 STATIONS AND ROUTE SECTIONS

4.1 *The Písnice station*

The Písnice station is very important both for the overall line D and the southern sector, but also, above all, for the onward transport services operating outside the agglomeration. This is because the station will become a large traffic terminal for buses and for car traffic. At the same time, the station is probably to be a three-rail terminal. Running rails continue further as a connecting track to the planned depot for the Line D in Písnice. This arrangement is suitable in the case of the Line D definitively terminating at this location. This cut and cover station, which is designed to have an intermediate platform and two surface concourses, is at a depth of 8.3 m under the ground surface. Connecting route section creates two single-track tunnels. The length of these tunnels is 1258 m.

4.2 *The Libuš station*

The position of the Libuš station is designed in compliance with the principles of the town planning scheme. It is in the neighbourhood of Novodvorská Street, across the Libuš residential area and its servicing facilities centre. The station is designed as a mined single-span structure, at a depth of about 25 m under the ground surface, with one at-grade concourse oriented toward the south, toward a pedestrian subway under Novodvorská Street.Route section in direction to the station Nové Dvory is formed by two single-track tunnels as well. The total length of these tunnels is 763 m.

4.3 *The Nové Dvory station*

This station is designed as a mined station, the top of rail (TR) about 34 m under the ground surface, with two concourses. It is of an underground, mined, single-span, two-concourse type, capable of functioning as a terminus. The northern, main concourse is connected to Durychova Street, which is a place where a transfer link to onward means of transport is provided. The southern concourse is oriented toward Chýnovská Street. It is designed as an at-grade structure, which will be easy to incorporate to the new urban structure in the future. Next part of metro line is formed by driven double-track tunnel except part of line adjacent to the stations with central platforms. The total length of this section is 1259 m.

4.4 *The Nemocnice Krč station*

The station is designed as a cut-and-cover structure, with an intermediate platform connected at the ends with concourses via vertical roads. Both concourses are designed as lightweight glazed pyramid-shaped pavilions. The station is 13–15.5 m deep under the surface. There are two short single-tract driven tunnels at the station. Next part is formed by double-track tunnel which is 550 metres long. Small thickness of overburden does not allow to perform driven double-track tunnel under the street Nad Havlem. For this reason 282 m of special single-track tunnels combination was designed in front of metro station Nádraží Krč.

4.5 *The Nádraží Krč station*

This is the only at-grade station on this line. It has side platforms and two at-grade concourses. It is built partially on a bridge structure spanning Kunratice Brook between the “Southern Connection” expressway and the area of the railway station Prague – Krč. Despite the fact that the station is likely to be the least exploited station of the Line D in terms of the volume of passenger traffic, its undisputed significance lies in the unquestionable importance of the transfer to railway in the context of the development of the Prague integrated transit system and gradual implementation of an interval urban railway traffic system.

The other, not less important town design aspect of the Nádraží Krč station is the fact that owing to the pair of opposite at-grade concourses and connected pedestrian subways (the northern subway is under the "Southern Connection" expressway, and the southern under the station yard) the metro station allows the interconnection of the area found north of the Southern Connection, the grassed areas around an original manor-house and along a stream, and areas south of the railway station, which are today completely isolated from each other. In this way the metro station will make the green areas accessible for pedestrians, thus it will help to overcome the existing urban barriers formed by a couple of nearly parallel line traffic structures. A glazed steel structure of the platform and connected concourses will provide a pleasant contrast with the other underground metro stations. Daylight penetrating to the platform, the possibility to watch surrounding greenery and immediately adjoining water surfaces from the metro car interior, as well as the night illumination of the grounds, should represent architectural aspects for overall cultivation of this, today "forgotten" corner of Prague. Behind the station there is a double track tunnel perform by cut and cover method Length of this section is 283 m. Section from street V Podzámčí to the Station Olbrachtova is formed by double-track tunnel. Length of this section is 872 m.

4.6 *The Olbrachtova station*

This is a mined single-vault station with a pair of side platforms. It is situated under Na Strži Street, with the top of rail (TR) level about 26 m under the ground surface. The only at-grade vestinbule hall is placed to the free space of the south-western quarter of the intersection of Jeremenkova, Olbrachtova and Na Strži Streets. Route section between Olbrachtova and Pankrác stations is formed by double-track tunnel. Rail connection to metro line C is inserted into this section. Rail branching forced itself the enlargement of tunnel cross section.

4.7 *The Pankrác station*

This single-vault mined station with side platforms and the TR level about 33 m under the ground surface is situated in the close vicinity of the cut-and-cover station existing on the Line C, in an area delimited on one side by the intersection of Na Pankráci and Na Strži Streets. The construction will create a passenger interchange node comprising the station on the Line C and the station on the Line D. Nearly the whole route section between Pankrác and Náměstí Bratří Synků is formed by double-track tunnel. Merely a short part in front of the Náměstí Bratří Synků station is formed by single-track tunnels. Access gallery length of 450 m enabling transport of tunneling equipment was designed in this section. This gallery will be exploited as a ventilation shaft in the final stage.

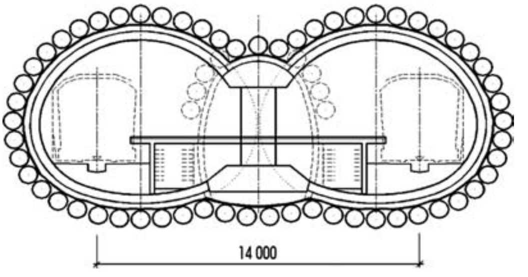

Figure 3. Cross section through the mined part of the Náměstí Bratří Synků station.

4.8 *The Náměstí Bratří Synků station*

This is a structure where combined techniques are to be utilised. The 30 m long part of the station that is situated at a shallow depth under existing buildings is an atypical double-vault mined structure with load bearing pillars in the centre of the platform. The station is situated to the flood plain of Botič Brook, and a major portion of the tunnel profile will extend into saturated fluvial sediments. A special microtunnelling technique is therefore being under consideration, i.e. a continuous canopy consisting of micro-tunnels installed along the profile of the future station tunnel to stabilize the subsequent partial excavation faces. The other part is situated outside the building. It is a large-span cut-and-cover structure with 2 rows of internal columns. The TR level is about 11 m under the ground surface. The station has side platforms and two concourses. The southern at-grade concourse is situated directly to the space of Bratří Synků Square.

Behind the northern part of this station short section of single-track tunnels was designed. Tunnelling of under Botič brook bed will be complicated for geological conditions which are very unfavourable. Water-bearing sediments of Botič valley wold and adjacent houses pose many problems for tunneling technology. Next part of route will be performed in open building pit. Remaining part of section between Náměstí Bratří Synků and Náměstí Míru stations form double-track tunnel and two single-track tunnels close to Náměstí Míru station.

Table 1. Basic information on the stations.

Station name	Station type	Platform depth under ground surface Variant 1 (2), m	Distance between stations, m
Písnice	cut-and-cover	8.3	
			1358
Libuš	driven*	25.0 (12.0)	
			863
Nové Dvory	driven	34.0 (19.0)	
			1359
Nemocnice Krč	cut-and-cover	13.0–15.5	
			910
Nádraží Krč	at-grade		
			1155
Olbrachtova	driven*	26.0 (14.0)	
			777
Pankrác	driven	33.0	
			1613
Nám. bratří Synků	combined	11.0	
			1153
Náměstí Míru	driven	40.0	
			1040
Hlavní nádraží	under the protective structure	15.0	

*Variant 2 allows cut-and-cover design for the stations.

4.9 *The Náměstí Míru station*

The Náměstí Míru station with the TR level at a depth of roughly 40 m under the ground surface is designed as a mined single-vault station with an intermediate platform, with direct transfer to the Line A, and with two concourses situated towards the points where Sázavská Street meets Vinohradská Avenue and Francouzská Street respectively. This design allows direct transfer from the Line D and Line A to all tramlines in the given area. The barrier-free access from the ground level to the platform of the Line D is provided by vertical lifts with their entrances in Korunní Avenue. The platforms on the lines D and A are also interconnected by a lift. Two single-track tunnels length of 940 m continue to the Hlavní nádraží station.

4.10 *The Hlavní Nádraží station*

One of the deciding traffic structures important for the whole city, which significantly affects the possibilities of making use of the area of reconstruction existing in the south-western quarter of the intersection of Seifertova and Italská Streets, is also the planned metro Line D, or the metro station Hlavní Nádraží. This station will be built under an inverted U-shaped protective structure. This structure will allow both the commercial buildings that are to be built in advance of the metro to be founded, and the metro station to be constructed subsequently inside this space. The client for the above ground part of the project will take care of technical feasibility of execution of openings in the protective structure required for the passage of vertical routes leading from the station (the northern concourse) and for the access ramps built for metro construction purposes.

Three driven single-track tunnels of stabling siding are part of this station.

5 DESCRIPTION OF THE VARIANT 2

The basic principle of this variant is the effort to get the light metro alignment as close to the surface as possible. The aim of utilization of smaller vehicles is to achieve a smaller clearance profile, steeper longitudinal gradients along the route (accommodation of the alignment to the terrain configuration so that the project is cheaper) and possibly automatic operation.

The spatial design of the stations is identical with the design used in the variant 1, although the stations are placed closer to the ground surface, to a position that is allowed by the maximum longitudinal gradient possible in the framework of the given terrain configuration. Regarding the Náměstí Míru and Náměstí Bratří Synků stations, even the elevation remains the same as the elevation used in the variant 1. The reason is that the Náměstí Bratří Synků station cannot be closer to the surface because of the existing structures in the underground space, and the priority for the Náměstí Míru station is to have short transfer relationships to the existing station on the Line A.

In general, from the aspect of the horizontal alignment designed in the variant 2, it can be stated that the alignment copies the trace of the variant 1, which means that the design parameters are fully compatible with a traditional metro system. Regarding the gradients, for the reason of virtual invariability of the elevation of the stations within the section Pankrác – Hlavní Nádraží, the route gradient is left identical with that designed in the variant 1. It is therefore again fully compatible with the traditional metro. Gradients over 40‰ are designed for the section Nové Dvory – Pankrác. This solution gets the Nové Dvory station closer to the ground surface approximately by 14 m (TR – 19 m, the station remains to be mined, three-vault type) and the Olbrachtova station roughly by 12 m (TR – 14 m, cut-and-cover design). The risen station Nové Dvory allowed also the Libuš station to get closer to the surface (TR – 12 m, cut-and-cover design). Conception of technical solution of track tunnels is the same as in variant 1.

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The Turecký Vrch railway tunnel

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ABSTRACT: The project of the modernisation of main railway corridors is in progress in Slovakia. The project includes the railway route Nové Mesto nad Váhom – Púchov. This route leads through a rugged country, which, in the Turecký Vrch section, is listed as a protected landscape area. Authors of the design studied several variants. The straightened alignment, and environmental reasons in particular eventually contributed to their preference to the tunnel variant over an up to 40 m deep side-hill cut leading along the entire length of the hill side. The mined double-rail tunnel is 1,775 m long, with the length of the mined section amounting to 1,740 m. The cut-and-cover sections total 35 m. The maximum overburden depth is 100 m. Complicated geological conditions, verified by an exploration gallery, were the reason why the authors of the design selected the New Austrian Tunnelling Method, which allows a flexible response to changes expected in geological conditions. The lining of the mined tunnel consists of two shells with an intermediate waterproofing membrane covering the vault and side walls. The tunnel profile is designed with regard to train aerodynamics, for the speed of 160 km/h (prospectively 200 km/h).

INTRODUCTION

At present, the railway line curves round the side of Turecký Vrch (Turecký Hill) in close proximity to a national road and the Biskupice canal. The current conditions in the narrow basin do not allow changes in the alignment which are required for increasing the speed limit over the track. This is why a new alignment has to be designed, with the best variant containing a mined double-rail tunnel. This variant provides the best solution to the conflict between the interest in protecting the Turecký Hill protected landscape area and the interest in the space for the modernisation of the railway line. The alignment passing through Turecký Hill begins in an extremely confined space in front of the entrance (southern) portal, where the railway line, the I/61 national road and a protective embankment of the Biskupice canal run in parallel with each other. It leaves the exit portal on the northern side via a right-hand curve, designed to minimise the work in the unfavourable environment of secondary loess. The portal proper can be built in a minimised construction trench.

The double-track railway tunnel is designed for the UIC C clearance profile, with firm trackbed and the track-centre distance of 4200 mm, on two R 2000 m and 1995 m reverse curves and a tangent section, and on minimum longitudinal gradients of 3.5‰ and 4‰; this design satisfies the speed up to 200 km/h. The clearance profile is unified for the entire length of the tunnel, with the exception of two contact-line tensioning chambers with an enlarged profile, which are approximately in the middle of the tunnel. The primary excavation support with a variable thickness of the lining will provide protection for a time sufficient for application of the waterproofing system and subsequent erection of the permanent secondary cast-in-situ reinforced concrete lining. Revetment walls are designed for both portal sections to allow the construction of the cut-and-cover tunnel blocks with a minimised impact on the surrounding protected landscape area. Natural and unforced incorporation of the portals into the landscape will be achieved by utilisation of free stone from a local quarry.

1 GEOLOGICAL CONDITIONS

The Turecký Hill massif consist mainly of carbonate rock layers and, partly, from the north, Quaternary sediments. The tunnel overburden depth varies from a minimum of about 3 m at the southern mined portal to approximately 100 m.

The northern (exit) portal and the about 80 m long initial section of the tunnel will be built in a complex of Quaternary sediments of aeolian and diluvial origin, which are represented mostly by strata of yellow to yellow-grey secondary loess. In the initial 40 m long section, the thickness of the series of strata is expected to cover the entire height of the tunnel cross section combined with polygenetic sediments consisting of brown-coloured aeolian-diluvial sandy shales. These sediments are frequently mixed with diluvial debris, which have an underlier consisting of about

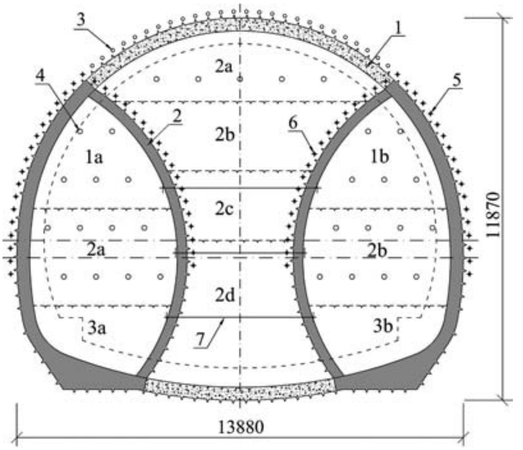

Figure 1. The cross section for the excavation through the Quaternary sediments. 1-Primary lining 400 mm thick, 2-Temporary lining 300 mm thick, 3-ø 108/16 tube micropiles or self-drilling micropiles injected with cement grout, 4-FACE SUPPORT: GRP anchors injected with cement grout, 5-Side drifts: GRP forepoles, 6-Side drifts: GRP self-drilling forepoles, 7-Tie rods.

2 m thick layer of Neogene claystone and sandy clay; the bedrock consists of dolomites weathered to the depth of about 2 m. The tunnel overburden is about 10 m to 15 m thick. An increased amount of diluvial debris mixed with loam, larger cobles and boulders is expected in the following 20 m long section, where the overburden thickness will increase to 25 m. In the remaining 20 m long section, the tunnel bottom will probably pass through the weathered bedrock, which will gradually rise to cover the entire tunnel cross section. The overburden will be 30 m to 35 m thick.

The other tunnel section, which is 60 m long, will be driven through a rock mass consisting of dolomite layers in the invert and bench, overlaid by a series of limestone layers in the excavation crown. Apart from several dip faults, there will also be effects of weathering encountered in this section; karst phenomena cannot be excluded in this section.

The rock massif through which the 1600 m long tunnelled sections will pass consists of Mesozoic series of layers of carbonate rocks (dolomites and limestone). Based on the results of surveys, we can state that the Mesozoic massif of Turecký Hill and mainly the dolomite series of strata are intensely tectonically disturbed. The blocks of carbonate rock are not affected by a more significant process of deep weathering. A larger depth of weathering may be bound to the faults; the weathering depth can be expected to vary between several meters and first tens of metres. Apart from sinkholes, the karst phenomena will even be present in subsurface parts of the massif. They will be mainly bound to the zones or more intense fracturing and loosening of the rock mass, or to the zones of tectonic failures and open faults. The carbonate massif is expected outside the zones of tectonic faults and rock loosening should be water saturated to a minimum degree. Despite the nearly continuous covering by impervious aeolian sediments, loess and secondary loess, ground water is supplied to the undisturbed blocks of the massif nearly exclusively through infiltration of precipitation. No significant impact on the existing regime of surface water or underground water due to the tunnel construction is expected.

2 THE DOUBLE-TRACK MINED TUNNEL

2.1 *Tunnel excavation*

The main proportion of the excavation is expected to be performed from the northern portal, only a smaller part will be driven from the southern portal. The reason is the large volume of earthmoving and large extent of the support of the construction trench required at the southern portal, problems with dump location in the vicinity of this portal, and the necessity for solving the crossing of the large-volume muck transport with the existing operating railway line. The drill-and-blast technique (controlled blasting) is expected to be utilised for breaking the competent rock. Soil and fractured rock mass will be disintegrated mechanically by excavators or even heavy impact breakers.

The New Austrian Tunnelling Method (the NATM) is designed to be utilised, with the primary support consisting of C 16/20 sprayed concrete, welded mesh, rock bolts, lattice girders and spiles. Based on the engineering geological survey, the following excavation support classes are anticipated: 2, 3, 4, 5a1 and 5a2 in the rock environment and 5b1 a 5b2 in the environment consisting of Quaternary sediments (a soil environment).

In terms of complexity and the geological conditions encountered, the tunnel being designed is categorised as geotechnical category 3, which belongs among the most difficult in the field of tunnel construction. The poor quality of the rock massif is a result of the existence of discontinuities, weathering, proximity of significant faults and possibility of occurrence of karst phenomena. The above-mentioned difficulties adversely affect not only the process of breaking the rock by the drill-and-blast technique, but also the process of the rock massif deformation lasting until a new state of equilibrium is reached. It will, therefore, be necessary, even with respect to the size of the excavated cross sections, to divide the excavation face in both directions, horizontal and vertical.

The excavated cross section for the soil environment (an 80 m long drive) is 13.9 m wide and 11.9 m high; the excavated cross-section area is 137.2 m^2. The tunnel profile is closed by an invert and is divided into two side-wall partial excavation faces and a central partial excavation face. The partial excavation faces are sub-divided into top headings and cores with bottoms. When both side-wall excavation sequences are finished, the excavation of the central pillar will start. When the entire soil environment tunnel excavation section is completed, the invert will be cast in parallel with the construction of the central tunnel drain and toe drains. When the required strength of the invert concrete is reached, a temporary roadway will be constructed and the tunnel excavation through the rock massif (toward the southern portal) will start. The primary lining thickness of 400 mm is designed for this environment, while the temporary lining of the central pillar excavation will be 300 mm thick. The tunnel length is divided into three sections differing in terms of the method of the crown support. The stability in the first section will be provided by a two-tier canopy of ø108/16 × 25 m long grouted pre-support tubes, which will be installed from the portal excavation trench. The second section, where the crown is found in relatively cohesive soil, will utilise a canopy consisting of fans of grouted self-drilling or driven spiles 6 m long, with 5 m overlaps. The third section lies at the soil/rock massif interface, where layers of little cohesive soil and debris may occur. For this reason, the canopy designed for this section consists of fans of ø108/16 × 10 m long grouted pre-support tubes with 5 m overlaps. All of the partial faces will be protected by a layer of shotcrete. The stability of the faces will be provided by glass fibre reinforced plastic anchors (a 40/5 mm GRP band grouted with cement mortar). Effectiveness and the necessity for these anchors will be verified in the first section of the excavation.

The excavated cross section of the tunnel driven in the rock environment (a 1660 m long drive) is 13.1 m to 13.7 m wide and 10.3 m to 11.4 m high. The cross-section area ranges from 105.1 m^2 to 129.7 m^2. A horizontal excavation sequence consisting of the top heading (the maximum height of 6.8 m) and the bench with the bottom is designed for the whole length of this section. This sequence meets the requirements for keeping the excavation face stable, as well as for accessibility and sufficient room for usual tunnelling equipment. The thickness of the primary lining is designed at 150 mm to 300 mm.

There are two 11 m long contact-line tensioning chambers with the excavated cross-section area of 194.4 m^2 designed at chainage 103.325 and 103.425. The enlargement to the profile designed for the chambers will be carried out subsequently when the entire tunnel length excavation is completed with the basic cross section maintained. The enlargement work will start in the top heading, then the remaining part of the profile will be enlarged. The chambers are provided with an invert and 400 mm-thick primary lining. There will be safety recesses on both sides of the tunnel, at intervals of 20 m.

Based on mathematical modelling, we expect that deformations of the primary lining in the crown in the support classes 2 to 4 will be up to 50 mm, in the class 5A up to 80 mm and in the class 5B up to 150 mm. There are no buildings inside the settlement trough. Contingent increased settlement cases pose no threat to the surrounding area; they are only a problem in terms of the tunnelling work.

2.2 *The final lining and tunnel equipment*

The final lining of the mined double-track tunnel will be constructed using travelling tunnel formwork. The casting will be divided into two phases: the invert or strip foundation casting and casting of the vault and side-walls. The casting blocks will be 10 m long. A minimum 2 m wide and 2.2 m high (measured from the walkway surface) safety recess is located in every other block. The recess contains an inspection shaft built on the drainage, and a manhole on the cable duct. Cable chambers for installation of the contact-line tensioning equipment will be cast in two phases, identically with the entire tunnel.

The internal radius of the final C25/30 reinforced concrete lining is 6.1 m. There are several variants of the lining designed, depending on the excavation support class and foundation conditions. The thickness of the lining reaches 300 mm to 400 mm. The reinforcement consists of lattice arches combined with welded mesh or strap pieces if necessary. The major part of the tunnel is founded on strip foundation; the 600 mm thick invert is designed only for the portal blocks and the section driven through soil. In both above-mentioned cases, tie-up reinforcement is designed. Blinding concrete and infill concrete is of the C 16/20 and C 8/10 grade. The intermediate waterproofing system is of the umbrella type, i.e. covering only the vault and side walls and ending at the level of the toe drains.

The central tunnel drainage duct consisting of a 500 mm-diameter PE pipe is embedded in the tunnel bed between the two rails. Toe drains consisting of 200 mm-diameter flat-bottom PVC drainage pipes are on both sides, behind the outer surface of the final lining. The toe drains pass through inspection manholes built in the safety recesses; transverse drains lead from each manhole to manholes on the central drainage duct. Both drainage pipelines are embedded in porous concrete, at a gradient identical with that of the railway track. They terminate in a pair of interconnected shafts, from which water flows to fire protection reservoirs.

Walkways about 900 mm wide are designed on either side of the tunnel. There are steel handrails installed at a height of 1.1 m above the pavements; the lines of the handrails are broken at the safety recesses. There is a 12-way PE cable duct embedded in the walkways. The cable duct is cut in the safety recesses by manholes containing outlets for electrical equipment (tunnel illumination, sockets, switches). The dry fire main (a 110 mm-diameter PE pipeline), which is fed with water from the fire protection reservoirs, is found under both walkways. A hydrant valve is installed in the inspection manhole in each safety recess. Both branches of the dry fire main are interconnected every 100 m; the connecting pipelines are terminated by outlet valves installed in shafts on the central drainage duct to make emptying of the fire main possible.

The numbers of all tunnel blocks, safety recesses and escape routes and their distance from the closest portal or the escape gallery entrance will be marked in the tunnel. There are oblique guide-strips interconnecting the neighbouring safety recesses designed with the aim of facilitation of safe movement and search for a shelter in the tunnel. The marking will be painted on the internal surface of the final lining. Sign tables will be imprinted into the concrete surface during the casting.

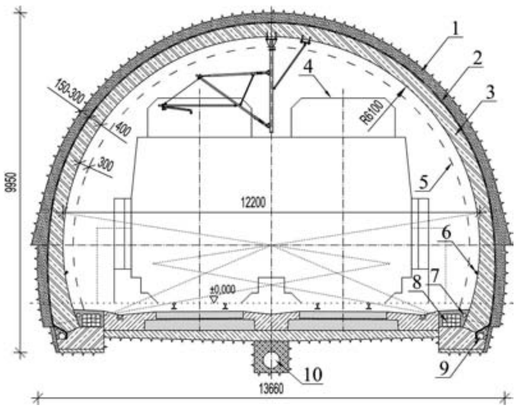

Figure 2. Typical tunnel cross section 1-primary lining C16/20 shotcrete, 2-waterproofing layers, 3-secondary lining C25/30 reinforced concrete, 4-safety margin, 5-clearance envelope, 6-steel handrail, 7-fire main, 8-cable duct, 9-drainage behind outer surface, 10-central drainage duct.

3 THE NORTHERN (EXIT) PORTAL

The exit portal is located in a deep erosion furrow in the northern slope of Turecký Hill. The terrain surface in front of the portal will be cut to comply with requirements of a mustering area for the fire rescue service equipment and emergency vehicles, including an access road with a bridge over Bošáčka stream. This area will be supported by retaining walls and provided with a handrail above the stream. There will be a small building housing a standby power unit and the tunnel equipment control elements there, and a fire protection reservoir, which is built in an excavation box lined with secant pile walls. The reservoir is fed with water from the stream, and also with the water from the tunnel drainage treated by means of a downflow baffle. The water from the reservoir flows again to the stream, via a safety overflow. The exit portal area is at a distance approximately of 100 m west of the existing railway track.

The construction of the cut-and-cover tunnel and a permanent revetment wall requires a 24 m × 32 m construction trench to be excavated and provided with temporary support. The rock massif in the area of the trench consists of a complex system of Quaternary sediments (see Geological conditions – the northern portal), which extends approximately 4 m under the tunnel structure. The water table is found roughly 7 m under the structure. The vertical walls of the construction trench will be supported by a system of vertical micropiles (ø152/18 tubes) provided with a layer of C20/25 sprayed concrete with welded mesh. The walls will be anchored at six anchoring tiers by 24 m to 30 m long stranded anchors (10 m-long roots). The tops of the micropiles are covered by anchored reinforced concrete capping pieces; in the lower tiers, the anchors pass through steel wallers. The anchoring of the reinforced concrete capping pieces skirting the terrain is permanent. There are parapet walls built on the top of the capping pieces, and gutters diverting rainwater flowing down the slope outside the portal area are laid behind the parapet walls. A pile wall which will be built from the level of 1.6 m above the top of rail will extend the northern micropile wall (parallel with the railway track), thus the maximum depth of the excavation will reach 18 m. The former pile wall consists of 1.20 m-diameter, 12.3 m long piles. The toe of each reinforced concrete pile is keyed into weathered dolomites; each pile is extended by means of three pieces of 8 m long micropiles (ø108/16 tubes). The top of each pile is anchored by a 34 m long temporary stranded anchor (a 10 m long root). The opposite southern pile wall, with the length equal to the length of the cut-and-cover tunnel, will consist of the same type of piles, 10.2 m long, with the tops at the level 0.5 m under the top of rail. The piles are again extended by micropiles; the tops are not anchored. The two opposite pile walls built along the cut-and-cover section will be braced against each other under the invert by an 800 mm thick vaulted reinforced concrete slab. The entire above-mentioned structural system is designed to transfer the load to the competent bedrock. In addition, a protective 500 mm thick reinforced concrete transition slab will be cast between the portal and railway bridge, which is designed to eliminate the differential settlement existing in this section.

The construction of the final lining in the cut-and-cover sections will start only when the casting of the adjacent blocks of the mined tunnels has been completed. The same travelling form is planned to be used for both the mined and cut-and-cover tunnel sections; formwork for moulding of the outer surface will be added in the cut-and-cover section. The cut-and-cover portal block is 10 m long; it is provided with the invert. The front end of the portal block is vertical. The thickness of the upper vault lining of 550 mm is uniform; the invert is 800 mm thick; tie-up reinforcement will be used. The tunnel equipment is identical with that used in the mined section.

A final reinforced concrete revetment wall (C 25/30) lining the excavation box will be built in parallel to the railway track in front of the entrance portal and perpendicularly to the track behind the portal. The walls provide permanent support to the slope in the environment consisting of Quaternary sediments which are liable to sliding. The walls are supported with permanent 34 m long stranded anchors installed in three tiers. An unreinforced concrete block originates around the portal block of the mined tunnel. The block is found between the revetment walls. Its surface, inclining at 5:1, is clad in natural stone obtained from a local quarry to be properly incorporated into the landscape.

4 THE SOUTHERN (ENTRANCE) PORTAL

The new alignment declines from the existing route toward the entrance portal at a very acute angle, and it runs along a side-hill cut in rock, which gradually grows in the depth. The slope of the cutting reposing at 40° to 60° is stabilised. The new alignment is at a 1 m distance only from the original route at the beginning of the cut; the distance reaches 16 m at the end. A temporary relocation of the track shifting the rails approximately 3 m from the existing track in the direction from the hill side is designed for the period of the works to allow enlargement of the site facility in front of the portal and increase the safety of operations during the course of the construction. The portal is located in a distinct erosion furrow in the eastern slope of the massif, which consists chiefly of carbonate series of strata containing the Wetterstein Limestone and, in the top layers, Quaternary sediments. A hard surfaced area for mustering of the fire rescue service and an access road with a bridge over the Biskupice canal will be provided in front of the portal. There will be a small building on the edge of this area, which will house a standby power unit and the tunnel equipment control elements. A cut-and-cover fire protection reservoir (partially covered with the backfill of the portal block) will be built beside the portal. The reservoir is supplied with rainwater collected in the erosion furrow and flowing via a trough and a storm-water inlet, and water from the tunnel drainage treated by means of a downflow baffle. The safety overflow is provided via an old culvert leading to the Biskupice canal.

Figure 3. The northern (exit) portal.

The entrance portal is part of the cut-and-cover tunnel section; it is built in a 25 m long excavation box, which is located in a side-hill cut with the rock wall about 17 m high and is an extension of the side-hill cut excavated in front of the portal. The vertical walls of the excavation box will be supported by micropiles (ø152/18 tubes) and C 16/20 sprayed concrete with welded mesh. The tops of the micropiles will be fixed in a reinforced concrete capping piece, which will be anchored with 6 m to 8 m long pre-stressed stranded steel rock anchors. The lower tier of anchors will pass through steel wallers. A canopy consisting of horizontal ø108/16 × 25 m long grouted pre-support tubes will be constructed at the front end of the excavation box to stabilise the excavation at the beginning of the tunnel drive, where the overburden will be very shallow and its transverse gradient is very steep.

The final lining of the cut-and-cover tunnel is constructed in the same way as in the case of the exit portal. Two cut-and-cover tunnel blocks have the invert. The front end of the portal block is slanted and provided with a flange preventing rainwater from flowing over the edge. The final liners of the vault and invert are 550 mm and 600 mm thick respectively. The tie-up reinforcement system is designed. The tunnel equipment is identical with the equipment used in the mined section. The backfill of the cut-and-cover tunnel plus the portal will be carried out approximately at a slope angle identical with that of the original slope. The stability of the slope will be improved by spreading a layer of natural stone.

The side of the 120 m long side-hill cut excavated in front of the portal will be supported by a revetment wall parallel with the rails; the height of the wall will increase in the direction toward the portal. The lower part of the wall will consist of one or two levels of a 500 mm thick masonry wall built using free stone selected in a local quarry. The wall is inclined at 5:1, and the individual levels are separated by a berm. The upper part of the revetment wall, which is separated by a berm lined with a handrail, is clad in grass pavers laid at a 70° incline. The total thickness of the structure amounts to 600 mm. In both above-mentioned cases, the surface of the side-hill cut is supported by shotcrete with welded mesh tied to the surface with anchors. Grass pavers will stabilised, if necessary, using 4 m to 6 m long corrosion-protected rockbolts. The part clad in natural stone will be stabilised by an anchoring system using the above-mentioned rockbolts. The berm is not designed for the lower part of the 10 m long section of the masonry revetment wall found just before the portal so that sufficient space is available in front of the portal.

5 THE ESCAPE GALLERY

The escape route, which is designed in compliance with the fire safety concept and results of the risk analysis, leads via a 244.7 m long transverse gallery driven on a 0.5% downward gradient from the tunnel and ends on the surface in the location of a former bridge over the railway track. The escape gallery has a horseshoe shaped profile. It will be driven by the NATM, with the support consisting of C 16/20 sprayed concrete primary lining, welded mesh, rock bolts, lattice arches and spiles. Based on the engineering geological survey, the design specifies 2 excavation support classes: class 2 and class 3.

The casting of the gallery lining is expected to be divided into two phases, the bottom and the vault plus side walls. The 120 mm thick bottom will be cast using C 25/30 concrete reinforced with welded mesh combined with tie-up reinforcement. The drainage sub-base will be made from porous concrete. The round vault with the inner radius of 1.65 m and the side walls will be made of C25/30 reinforced concrete with the minimum thickness of 200 mm. The main reinforcement elements are lattice girders and welded mesh. The intermediate waterproofing membrane covers the vault and side walls, and ends at the bottom in the longitudinal toe drainage.

A 17 m long air lock chamber is provided in the part of the gallery adjacent to the tunnel. It is separated from the tunnel by a 2.2 m wide fire resistant door, and from the escape gallery by a 2.2 m wide steel door. The air lock is positively pressurised by a forced ventilation system independent of the escape gallery ventilation system to prevent smoke entry from the tunnel to the escape gallery.

The rock wall created on the surface in the location of the escape gallery mouth is supported by a 300 mm thick retaining wall provided with side wings.

6 CONCLUSION

The process of underground construction significantly differs from the other construction branches due to the specific nature, limited knowledge of geological conditions, often unpredictable circumstances, dependence on the means and methods of construction, and significant construction risks. The application of the NATM makes the construction process as well as the completed structure safe through the observational method and thorough geomonitoring. Geology is the factor which affects the complexity, technique and duration of construction. It is best visible at the northern portal where the Quaternary sediments represented by secondary loess significantly complicate the work on the excavation box, tunnel excavation and erection of the final lining. The most serious complication will be the task to excavate the side-hill cut in rock in the constrained conditions existing at the southern portal, in close proximity to the existing railway track, which will be in service throughout the construction period. Despite all problems, the design tries to respect the character of the natural forest-steppe environment in the protected landscape area of Turecký Hill.

Regarding the construction organisation, the construction time could be shortened if necessary by implementation of additional one or two points of attack in the middle of the tunnel, at the end of the escape gallery, which would be driven and provided with primary lining in advance. This solution would, however, require widening of the access road to the escape gallery and possibly even enlargement of the gallery cross section according to the tunnelling equipment used.

The detailed design for the Turecký Hill tunnel was issued in 2005. In Slovakia, it is a kind of a "pilot design" developed within the framework of the "Upgrade of Railway Lines of the Slovak Republic to speeds up to 160 km/h", prospectively 200 km/h. The completion of the Turecký Vrch tunnel will add an important section to the quality and fast link between Bratislava, the Slovak capital, and Žilina, a centre of northwestern Slovakia. The tunnel construction is expected to start in 2007.

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Collectors – System approach to the regeneration of technical infrastructure in the city of Prague

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ABSTRACT: Collectors represent a new element in the quality of distribution and regeneration of technical infrastructure, which in a long-term perspective enables its inspection and maintenance as well as the increase in capacity without negatively affecting the environment. For their successful spatial integration into the city, it is necessary to asses closely a set of limiting factors. These factors not only include the geological conditions, but also the existing buildings, technical infrastructure, traffic and city operation. New collector ducts generate an open system, which enables their further expansion. Their concept must respect the principles of underground urbanism and requires consistent coordination with other underground structures in the area with respect to land-use planning.

1 INTRODUCTION

Historical centre of Prague, the capital city of the Czech Republic is a collection of substantial cultural, architectural and urban values that developed over the last centuries. Its high significance is proved by the registration of the Prague historical reserve among the city zones that are protected by the UNESCO organization. However, an increasing number of the city's and country's social and cultural activities is based in the centre, which creates a great demand for traffic and technical infrastructure with prevailing citywide character. Every activity is dependent upon the infrastructure's full functionality and sufficient capacity.

For attractiveness of the city centre and development of its operation and service and due to the lack of surface space, a new resource for further development of the city is the underground. Many linear and solitary structures of various functional uses are situated in the underground. These involve a complex solution for technical infrastructure regeneration in terms of aggregation of two or more underground service ducts into collectors.

2 COLLECTORS – PRINCIPLES AND REASONS

2.1 *Structures' characteristics and their reasons*

For more than 30 years, laying of pipes and cables is being designed into underground man-sized ducts, mainly in new residential areas in the suburbs of Prague. This way of underground services laying has a significant and substantial role in the regeneration and development of technical infrastructure within the Prague historical reserve. Existing cable and especially pipe service ducts are often old and considerably defective, very often with reduced or exceeded lifetime. Moreover, with increasing demands on the city development, their capacity becomes insufficient.

A gradually updated plan of collectorisation of the municipality is being created for the purpose of land-use planning. According to this plan, an open system of collector ducts is being created. This system enables further expansion into other parts of town and branching-off into adjacent streets. Therefore, this is not only a local solution of technical infrastructure laying or realignment but a system solution of problems with technical infrastructure in city centres.

2.2 *Advantages of building collectors*

Advantages that result from laying of underground services into collectors are stated in the following notes. The advantages are from a long-term perspective obvious especially for consumption collectors that immediately supply particular surface buildings:

– Main aspect represents an easy inspection and maintenance of inbuilt utility lines without an impact on the surface, as well as the safety of the operation of the utility lines secured by a system of control measurements and centralized control of operation from one control centre.
– Increase in lifetime of utility lines by diminishing the negative effects of ground environment (corrosion, stray currents, dynamic effects of traffic etc.).

This relates to economical operation and a reduction in losses during faster search for damaged utility lines.
- Elimination of surface excavations for all subsequent maintenance and repair works on the utility lines. This results in a cut-down in traffic limitations as well as environmental impact.
- Possibility of easy increase in capacity of particular medium by laying a utility line of a greater size or even a change of a medium.

3 INITIALIZATION OF BUILDING COLLECTORS

The impulse for building collectors is a need for an analysis of possible solutions for sets of problems linked to the development of the surface and the underground of the city centre. Assessments of present conditions and prognosis of the future development are monitored in areas involving city territory and surface buildings, technical and traffic infrastructure and environment.

3.1 *Development of city territory and surface buildings*

This area includes assessment of complex development of a city zone or reconstruction of a city block. This also involves territorial preparations of locally situationally defined large solitary structures. A dispositional utilization of underground levels including the solution for regeneration of underground services must form an integral part of the historical district's land-use plan.

3.2 *Condition and development of underground services*

A Large amount of cable and pipe utility lines is in dissatisfactory state. Pipe ducts that are 80–100 years old can be considered in emergency conditions and over their lifetime limit. Outflow of water into subsoil is a common occurrence. However, even for relatively newer utility lines, negative effects of ground environment (humidity, corrosion, stray currents, dynamic effects of traffic etc.) contribute to shortening of lifetime. With the problem of lifetime and technical condition also comes the problem of insufficient capacity of existing utility lines for their extensive use as well as absence of sufficient reserve for their perspective development.

As an alternative to laying utility lines into the ground and persistent excavation works, a system of collector network has been under development in Prague since 1985.

3.3 *Traffic infrastructure solution*

Due to constantly rising traffic demands in the city centre, new solutions that enable both road capacity increase and solutions for present problem sites are being searched for. Conception of rail and non-rail traffic must respect specifics of the historical city centre and conditions of the city historical reserve, which limits the increase of traffic in the centre. A motivation for solving the problems of utility lines is also the modernization of tram tracks (including solutions for traffic narrows, which are very common in the city centre).

One of the solutions is the use of city underground levels, which are mainly used by linear structures (metro, road tunnels). This incites the need for solutions for exposed traffic junctions by building overpasses and underground garages. This usually incites the need for realignment of great number of utility lines (preferably into matched structures – collectors).

3.4 *Environment*

Construction works that relate to utility lines constantly disturb the environment of the city centre. These construction works are either intended (during expansion or renovation of utility lines) or unintended (during emergency). They always cause a negative impact on the city. Therefore, collectors represent an alternative solution for utility lines regeneration.

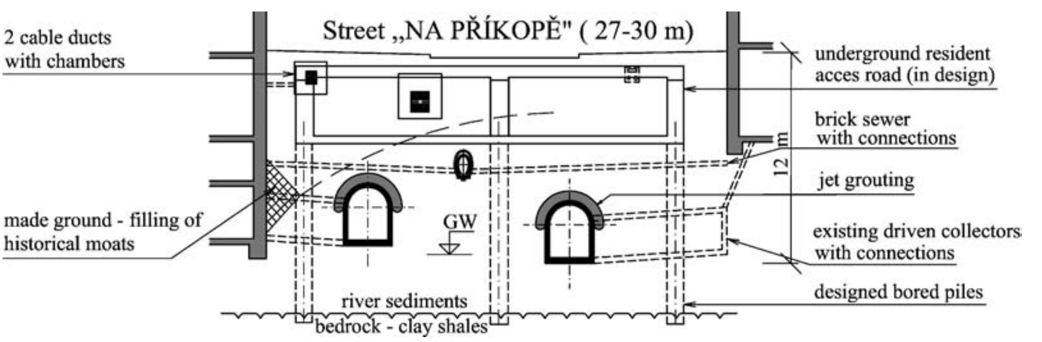

Figure 1. Prikopy Collector – prerequisite for utility lines realignment and possible future execution of an underground road.

Their execution represents a short-term environmental impact. Long-term positive effects will appear during their operation over the whole lifetime period.

4 CATEGORISATION OF COLLECTORS AND THEIR APPLICATION IN PRAGUE CENTER

4.1 *Philosophy of development of collectors in Prague*

A concept for systematic approach in collector network generation that represents a system solution for underground services in the centre of Prague was executed in a land-use planning documentation called "General plan of collectorisation in central Prague". The first concept was published in 1982 and covered a broader territory of the centre with an estimated development timescale of few decades. It was updated several times during the subsequent years to reflect new standards in the concept of the city and underground services development. The extent of collectorised territory was then partly modified to cover the area of "The Prague Historical Reserve". The last study, which specified the changes in the land-use plan, was completed in 2005. Its purpose is to include the main supply 2nd category collector ducts (see below) into the land-use plan – a basis for area development.

At present, the total length of operational collector ducts in Prague is more than 90 km. Out of this length, almost 18 km of operational collector ducts of both further described categories are in the historical centre. Other collectors are being constructed or are in an advanced phase of designing. The rest of about 80% of underground reinforced concrete collectors represent structures in new residential areas in the suburbs of Prague.

A Collector Control Centre at Senovazne Square is a part of the system and is the only control centre in the network. Constant operational monitoring and control of the branched collector system of the whole Prague (centre and suburbs) is performed there.

4.2 *Collector types*

Collectors are divided into two separate, but technologically connected systems according to their functional purpose. Besides the below stated character of inbuilt utility lines, both systems differ in situational and depth level positioning as well as cross section, which results in differentiation of applied construction technologies caused by geological conditions:

– 2nd category collectors – supply – essential distribution of utility lines of city-wide character in greater depths and wider territory with surface outlets in limited number of locations and blocks of houses. These ducts are intended to transport media into individual city zones. They lead straight and if possible in bedrock. Branch ducts, which end in shafts in duct crossings, connect deep technical chambers on 2nd category collectors with 3rd category collectors.
– 3rd category collectors – consumption – direct supply of adjacent buildings with particular media. Their situational and depth level positioning is close to place of consumption – adjacent buildings. They therefore usually lead on city owned land. Connection into the buildings is done by a side collector and then by borings for pipes and cables into basements of the buildings. Depth level of the collector, as close to the surface as possible, significantly affects the choice of suitable construction technologies. Influence of construction works on this type of collectors is highly notable on the surrounding area – especially on buildings and utility lines. This usually invokes big geotechnical challenges like acceptable deformation and the effect of driving on the surrounding area.

5 ASPECTS OF COLLECTORS' SITUATION INTO THE CITY

A complex assessment of all limiting aspects of collector construction as well as all specific conditions according to the characteristics of the place and structure is necessary for spatial situation of the underground structure into cramped situational and depth level conditions of the city centre.

5.1 *Surface structures*

Existing surface and underground structures represent a basic input for the solution. Definition of extent of surface structures in relation to public land is necessary for positioning collectors and all their structures, which are in contact with the surface.

For ducts in smaller depths, underground parts which stick out from facades into the area of the public

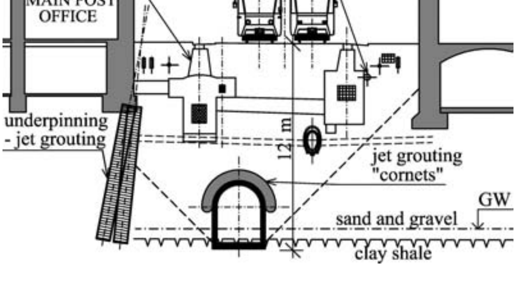

Figure 2. Design of most driven 3rd category collectors in the centre of Prague, under the level of sewer ducts.

land are crucial, as well as all underground premises of other structures. Unknown remains of filled and unfilled underground spaces and relicts of former area redevelopment are crucial, too. Logical demands for historical monuments preservation and archaeological research are respected when situating collectors in the town historical reserve.

From a geotechnical point of view, the character and quality of adjacent surface structures is crucial, as well as their static and constructive systems. Static securing of the structures before start of construction might be necessary.

5.2 *Existing underground services*

Utility lines are an issue for all underground structures in Prague. Search for pipe and cable utility lines of all owners in the area is crucial, as well as definition of inviolable ones (cannot be realigned), possibly realignable and realignable without problems. The inviolable utility lines of citywide character include trunk sewers, large pipelines and telecommunication cables matched into cable ducts with frequent chambers.

It is usually impossible to respect normative protective zones of all utility lines in the city centre. It is therefore necessary to adopt all protective and securing measures before and during the construction.

Another factor which comes into play is the demand by utility lines owners for full functionality of their existing lines during realignment. Therefore, the new realigned utility lines must be built in a different place.

Acceptable levelling of underground collectors is an issue as well. This is significantly limited by levelling of other media (especially sewers and cable ducts).

Levelling of sewer tubing (including sewer connection inlets) in depths of 6–7 m under the surface divides the subsoil into two levels.

This affects the possible levelling of the collector, either above or below the sewer.

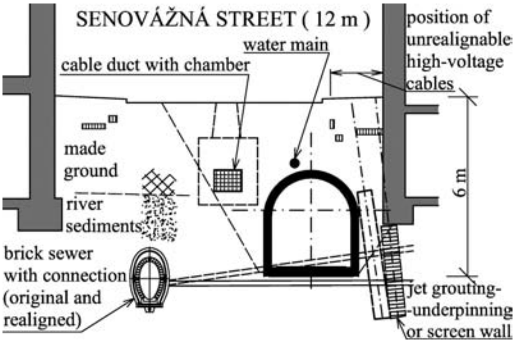

Figure 3. 3rd category collector whose execution brought up difficulties with utility lines and adjacent buildings supporting.

The collector must not be situated in the protective zone of the sewer and affect the structures of the sewer. The levelling is therefore possible either above the sewer (4–5.5 m under the surface) or below the structures of the sewer. Levels between 10–14 m below the surface are most common for collectors in the centre. Their least statically suitable levelling is due to execution in cohesionless soils, which are partly saturated. Position below the level of footing bottoms of most buildings demands complex static measures for securing the overburden and the adjacent structures.

5.3 *Traffic and city operation*

Traffic and city operation represents the third basic requirement for incorporation of collectors into the city. This is a very delicate problem not only during the concept phase but also during the execution phase of the collector, when it usually affects the public. Minimization of local negative effects on the operation of the city (its traffic and supply) and their duration is therefore demanded. Basic demand is functionality of traffic and rail mass transport. Construction works in protective zones of the metro are subject to special reviews and execution procedures. There is a strict requirement for structural and static inviolability of all metro structures.

Compromise solutions for possible temporary traffic replacements, which are acceptable by traffic and city authorities, are being searched for during the documentation reviews as well as solutions for construction site traffic (including acquisition of public land for construction site installations).

Enabling access to existing buildings and commercial objects in them (possibly by an alternative entrance) represents another measure during the construction works. Fulfilling of this demand is usually bound to applied construction technology, work processes and phasing.

In some cases, construction works on the collector might closely relate to works on a traffic structure. For example a parallel alignment of a collector and a traffic tunnel or underground collector and a tram track.

5.4 *Possibilities of execution*

Selection of possible application of specific technologies for collector execution depends on spatial resources of the site, geological conditions and assessment of impact of the works on the vicinity. Cut and cover technology and pre-cast constructions are mainly used for collectors in the new residential zones in the suburbs.

On the other hand, in town centres, feasibility of execution of cut and cover linear structures is restricted to entrance shafts in crossings and branching of collector ducts, as the effect of construction works on the city operation and environment is substantial in the centre.

Underground tunnels for particular collector ducts are driven from these shafts, which represent initial structures. Differentiation of used technologies, procedures and supporting measures is determined by the collector category and therefore its levelling (depth under the surface).

3rd category collectors are designed in small depths. They therefore require large amount of additional measures for stability securing and safety of boring, as well as static supporting of adjacent structures and underground services. 2nd category collectors are driven deeper, usually in bedrock, which is, if possible, unaffected by weathering.

5.5 *Collector preparation specifics*

This issue is also important for incorporation of collectors into the city. Public benefit of these structures is not that obvious, when all the utility lines situated in the ground are still functional. Moreover, expansion of the metro is more appreciable by the public and city authorities.

Specifics of preparation mainly result from great amount of parties to an action during the documentation reviewing at every level of administrative action. Property ownership matters of adjacent buildings are difficult. This incites the need for permits of all the owners (mainly for connections into their buildings). Present standard practice of the construction process creates harder relations between the involved partners. Demands for responsibility for damage done to others' property and the environment are applied consequentially. Complex evaluation of the present state of all structures before the start of works is necessary for further negotiations about damage caused during execution of the collector. This is performed by monitoring of present state of the buildings and sewer ducts as well as geotechnical monitoring during the execution performed on the surface as well as in the underground.

6 GEOTECHNICAL PROBLEMATICS OF DESIGN AND EXECUTION

6.1 *Solving geotechnical tasks*

For incorporation of a collector into the city, in addition to the above stated spatial aspects, it is necessary to work out a geotechnical analysis of static behaviour of the structure and its effect on the surrounding environment. It is necessary to review the process of execution of the collector during all construction phases as well as effect of the structure during the whole life time. Solving the geotechnical tasks, is therefore one of the limiting factors of the conceptual solution in which both limit states (ultimate, serviceability) must be respected.

The effect of construction works on the progression of strains in bedrock and the creation of subsidence zone is monitored with respect to safety system "collector structure – bedrock environment – buildings and underground services". Disposition, design, technology and process of execution, as well as a set of measures as a part of advanced works or driving of the collector arise from analysis of acceptable strains of surface structures and utility lines in overburden.

Consistent evaluation of static aspects of possible levelling of the collector is an integral part of these analyses. This was closely described in the categorization paragraph.

6.2 *Rock conditions*

Local geological conditions are a basic input data for geotechnical analysis. Their complexity in the centre of Prague is not only caused by historical development and broken morphology, but also by lithological evolution of rock environment and tectonic deformation of bedrock. Another factor is the erosion caused by the Vltava River. Two characteristic units can be defined in the geological profile – bedrock and quaternary sediments. Bedrock is situated in depth between 12 and 15 m under the surface level, in areas with terraced levels, the depths drop down to 6 m.

Bedrock usually consists of Ordovician shale of various compositions (clay, sand, silt) with insets of solid layers of sandstone and quartzite. Differentiation of separate layers of shale causes big differences in physical and mechanical characteristics, as well as in weathering processes resistance.

Overburden layers of quaternary terraced sediments are formed by clayey sands and sands with gravel, which turn into gravels at the base of the quaternary. Specifics of the made-up ground result from their variable thickness and composition affected by anthropogenic processes. Known and unknown underground spaces from former buildings are common. Usual thickness varies between 3 and 4 m, in places with filled moats up to 6–7 m. Made-up ground is relatively compact, with common archaeological findings.

6.3 *Hydrological conditions*

Hydrological conditions are another important input, which directly affects the phases of the concept and execution of the collector. Groundwater table, which creates a continuous level in a permeable environment of gravelous sand and especially gravel, runs approximately between 10 and 12.5 m below the surface in a depth range depending on the terrain morphology.

Thickness of saturated layer varies between 1 and 5 m depending on the shape of the bedrock. Groundwater level and its changes over time depend upon the water level in the Vltava River. It was necessary to revalue the levels of groundwater used for designing collectors after the floods in 2002 and to raise the

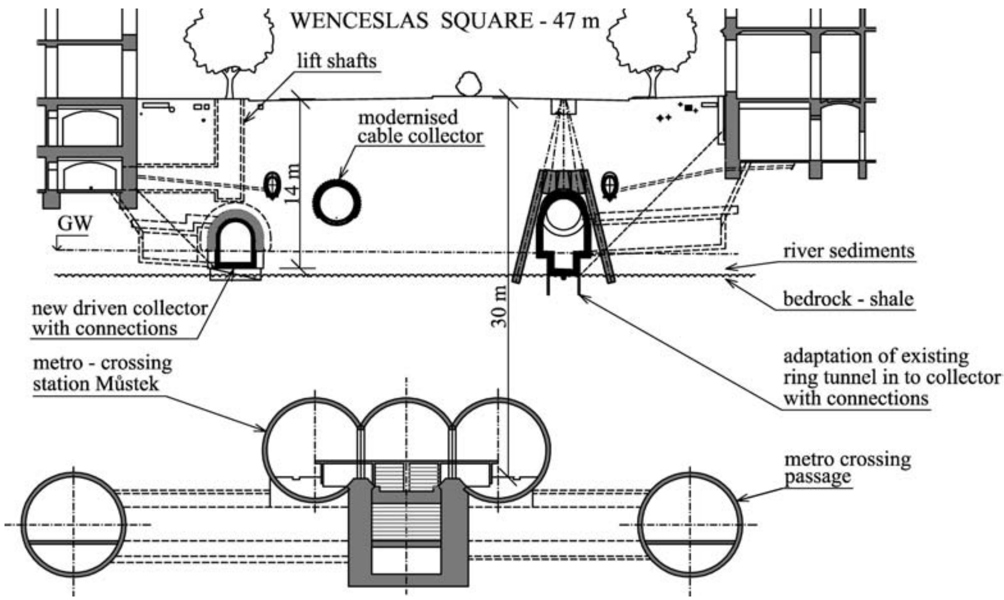

Figure 4. Further development of collectors in Prague – Collector at Wenceslas Square. Current execution of a new duct and proposed difficult adaptation of a present duct into a collector with connections into all adjacent buildings.

values of cent-year's flood level. Analyses of environment permeability, yield of water inflows and groundwater chemistry are necessary for determining the effect of the construction works on the area. A basic task is monitoring the behaviour of groundwater and its acceptable changes with respect to existing buildings.

7 COLLECTORS A UNDERGROUND URBANISM

Underground space has a great value in cramped conditions of the city centre for possibility of its utilization for purposes, which can't be placed on the surface or which would significantly affect the environment. There is therefore a necessary demand for systematic solution of proposed and planned underground structures including urban plans for underground utilization. Plans for utility lines development are an integral part of these solutions. Open system of collectors with the possibility of expansion is an appropriate solution in the centre of Prague.

Consistent coordination of all structures including qualified prognoses of future solutions and procedures is necessary, so that the planned structure isn't limited or restrained by previous inappropriate actions. The aim is to eliminate spontaneity and collisions in the underground utilization and to establish a system, similar to the existing system of urban plans of the surface. In case of collector network, this mainly means defining the levelling and solving collisions with metro and main utility lines of citywide character.

This also involves a sense of permanent value of all underground structures and their difficult dismantling. These financially demanding structures (usually with long pay-off period) are designed for long time operation (in terms of decades). Complex conception of the task of spatial incorporation of the structure demands a multidisciplinary approach of all participants.

8 PROGNOSIS OF FURTHER DEVELOPMENT

Development of collectorisation in Prague will continue. More than 3 km of consumption collectors are being built or prepared. Expansion of deep supply collectors (including an underground crossing of the Vltava river) in total length of more than 3.2 km is being prepared at the same time.

Operating collectors, as well as collectors in preparation, contribute to valuation of existing facilities in the area, improvement of operation of the city and its environment within the historical centre. Therefore, it is important to include the system of collectors among the components forming the city.

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Years of planning and implementation: Two major urban tunneling projects in Los Angeles, California, USA

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ABSTRACT: One of the world's major metropolitan areas, Los Angeles, California, has seen construction of over 80 km (50 miles) of large diameter tunnels for subway, wastewater, and water supply over the last 25 years. Two of these tunneling projects, the East Central Interceptor Sewer (ECIS), and the Los Angeles Metro Gold Line Eastside Extension (MGLEE) are examined to describe the 15 to 20 year durations of planning, design, and construction of the tunnels and to provide an overview for future planning and scheduling in urban settings. Aspects of the projects contributing to the complexity and schedule are discussed, including; identification of need and initial planning, the environmental processes, design and changes in technology, impacts of public perceptions on tunneling, protection of property, funding, and changes in all of the above over the duration of the project. The two projects are very different in terms of ultimate use. The complex process planning and design for tunneling in urban areas, however, are similar.

1 TUNNELS IN LOS ANGELES – OVERVIEW

Over 80 km of large diameter tunnels have been constructed in Los Angeles, California (USA) over the past 20 years for transportation and water projects. Project duration, including construction, planning, and design can be well over about 15 years for these tunnel projects in urban areas. In this paper the Metropolitan Transportation Authority's (Metro) Gold Line Eastside Extension (MGLEE), and the City of Los Angeles' East Central Interceptor Sewer (ECIS) are described.

The recently complete ECIS project involved over 18 km of Earth Pressure Balanced (EPB) driven tunnel using four TBMs through a densely populated area of central Los Angeles. The MGLEE tunnels are under construction as of September 2006 and represent 4.3 km of underground construction within a 10 km light rail project, also in a densely populated urban area of East Los Angeles. Two EPB TBMs are being used to drive the MGLEE tunnels.

Both tunnel projects are successful in terms of tunneling performance. Although the two projects are very different in terms of ultimate use; the complex process of tunneling in urban areas in the United States is similar. The following provides short summaries of project history for use in future planning of urban tunneling projects. As these two projects were constructed along the same approximate timeline (over 15 years), we also present some of the relationships between the projects; for example, changes in technology and local perceptions of tunneling. Other projects in Los Angeles are presented supporting case history.

2 METRO GOLD LINE EASTSIDE EXTENSION

The MGLEE tunnels have been in planning, design, and construction for over 20 years. Figure 1, Project Timelines, illustrates the overall schedules. The MGLEE project will extend the existing Metro Gold Line Light Rail (LRT) presently in operation from Pasadena to Union Station in downtown Los Angeles. MGLEE includes the construction of a dual track system, in three cross-sections: Aerial (3 bridges), tunnel (2.7 km) and at-Grade (7.1 km). The underground segment was placed under narrower sections of 1st Street in the older Boyle Heights neighborhood of East Los Angeles. Underground structures include two portals, cut-and-cover sections adjacent to portals, twin-bored tunnels (6.7 m diameter), and two stations.

Upon completion, the LRT will extend the Metro Rail system, currently comprised about 118 km of

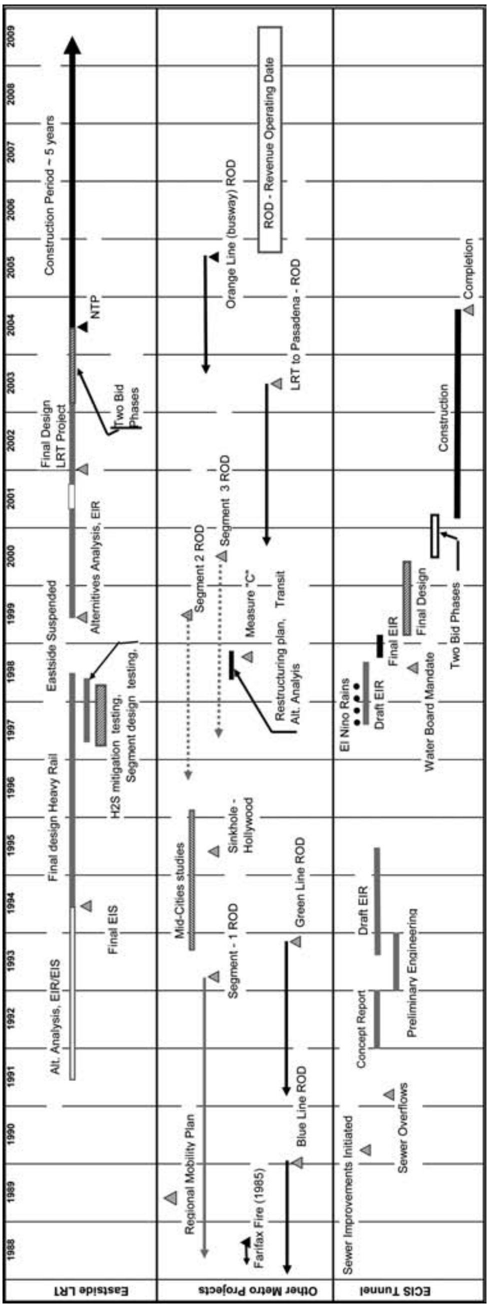

Figure 1. Project timelines.

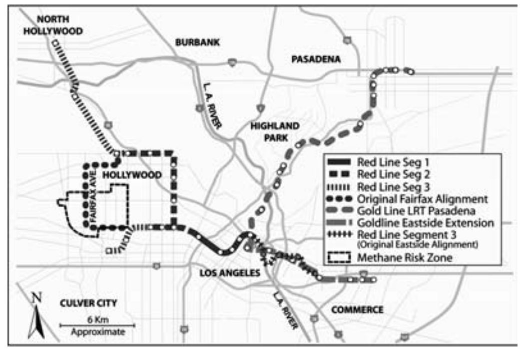

Figure 2. Metro Rail Systems, existing and planned (Blue and Green Line LRTs not shown).

urban rail and 22 km of busway. Figure 2, Metro Rail Systems, Existing and Planned, illustrates the MGLEE alignment relative to the subway system, and previously planned Metro segments. The existing Metro system includes three LRT lines (Blue, Green and Gold), the first becoming operational in 1990).

2.1 *Planning and environmental issues*

To understand some of the planning issues for the MGLEE project, a brief history of Metro Rail is presented. After two failed ballot measures in 1968 and 1974, Los Angeles County voters approved Proposition A in 1980, which authorized a half-cent sales tax to build the regional rail transit system. The Draft Environmental Impact Statement (EIS) concluded that a Rapid Transit System (ultimately heavy rail) would provide a high-capacity system for the regional core. The final EIS (1983) presented a 30 km subway from downtown Los Angeles west along Wilshire Blvd., and north on Fairfax Ave. (USDOT 2002). A methane explosion occurred in 1985 under a building in the Fairfax area, and subsequent studies recommended that "methane zones" be created in Los Angeles, requiring building code changes for construction in these areas. Tunneling was deemed unsafe, and Metro began studying alternative alignments to avoid the methane zone along Wilshire Blvd.

Metro re-aligned the subway or "Red Line" system to avoid the methane zone, and began design and construction of the system in three Major Operating Segments (1, 2 and 3) for funding purposes and to deliver completed operating segments in series. Segment 3 originally included plans for an Eastside Extension of the Redline Subway, a Mid-City Extension, and a Northern route. Of these Segments-3 routes, only the northern extension has been completed to date. The Mid-City Extension began planning to circumvent the methane zone.

Planning for the Eastside began in 1989 – with a Regional Mobility plan that formally identified the need for major rail transit improvements in the region and Metro's and the adoption of the 30-year Integrated Transportation Plan (Metro 1994).

Environmental Phases for projects in the USA were established after the National Environmental Policy Act (NEPA) became federal law in 1970. NEPA requires that alternatives be evaluated and subjected to public review. The preferred alternative(s) are carried forward for further environmental review during preparation of Draft and Final Environmental Impact Statements (EIS), with public comments addressed during the Draft phase. The state of California has similar requirements under the California Environmental Quality Act (CEQA), resulting in preparation of an Environmental Impact Report (EIR).

In April 1993, Metro and the Federal Transportation Authority (FTA) completed the Alternatives Analyses evaluate the Eastside Options. The preferred alignment would use heavy rail technology and continue Red Line Segment-1 from Union Station approximately 11 km east in two phases, the first being a 6.4 km tunneled segment. The Final EIS/EIR was completed in 1994 and final engineering began in early 1995, nearly in parallel with the Mid-City Extension.

2.2 *Design and technology Red Line Eastside extension*

Los Angeles geology presents some additional tunneling "constraints," namely seismic conditions and the presence of naturally occurring subsurface gas. Both conditions require additional provisions for design, construction, and final operations.

For all Red Line segments, a "two-pass" tunnel lining system was developed to include a "high-density polyethylene" (HDPE) water and gas barrier. Underground stations were also constructed using the HDPE barriers to "wrap" all underground structures. Open face shields were used for all soft-ground tunnels, as this was the traditional tunneling method used in Los Angeles, and most of the tunnel reaches were above groundwater.

Initial design for the Eastside Red Line tunnels included new challenges for Metro – given construction issues on prior tunnels – as well as discovery of new seismic feature and subsurface contamination, in addition to the overall seismic and naturally occurring gas conditions.

2.2.1 *Tunnels*

Final tunnel design began just as the Metro Red Line System construction had suffered from some highly publicized events: ground surface settlement of up to 30 mm was experienced along Hollywood Blvd's "Walk of Fame," (1994) and a sinkhole (1995) also in Hollywood, occurred during the enlarging of a Segment 2 tunnel to correct tunnel alignment.

Much of the planned Eastside subway length was non-linear, and meant that tunnel reaches were often under commercial and residential properties – as well as city streets. Tunneling risks were "elevated" due to recent experiences and the added potential property impacts.

In part due to publicized events, Metro commissioned an expert Tunnel Advisory Panel to assess tunneling conditions in Los Angeles with respect to world-wide experience. The Panel concluded in November 1995 that compared to world-wide experience, geologic conditions in Los Angeles were generally more favorable, the cost of tunneling relatively low, and tunneling performance, based on surface settlements, was "slightly better." The panel recommended consideration of EPB TBMs for future projects (Eisenstein et. al., 1995).

Designers were already recommending pressure-face TBMs, and slurry TBMs exclusively, for the Mid-City alignment. In Mid-City, additional geotechnical investigations were evaluating subsurface gas conditions – which were found to be as high as those in the methane zone, and with the addition of high hydrogen sulfide (H2S) levels. Studies for Mid-City concluded that not only should the gas-bearing formations be avoided, but also recommended consideration of ground pre-treatment, such as use of an oxidizing agent, and that tunnel construction should proceed using slurry TBMs to further contain gases in pipelines until they could be treated at the surface.

The studies on the Mid-City segment became applicable to the Eastside Extension, as ground contamination was found in an industrial area west of the Los Angeles River. H_2S and methane were present which lead to specification of slurry machines for this reach only. For other reaches, the Contractor would be given an option for pressure-face technology (EPB or slurry). Soil-gas studies for the Eastside included pre-treatment of the ground.

Use of pressure-face TBMs would also reduce the potential for surface settlement – as compared to open shield methods. Given that the alignment was not consistently within the public right-of-way, many buildings would be directly over the tunnels. Designers evaluated over 200 structures near or over the tunnel and developed a comprehensive compaction grouting program for additional building protection.

2.2.2 *Linings*

In addition to precautions taken for tunnel construction in gassy ground, tunnel final liners had to be "gas-proof." The traditional two-pass system with expanded initial liners would not be acceptable with the use of pressure-face machines, since effective use of pressure-face tunneling also requires gasketed segments and grouting immediately after the rings are assembled. No precedent for one-pass liners in gassy ground was found, and Metro undertook further studies in 1997–98 to develop a one-pass liner with additional gas sealing properties.

This one pass liner consisted of a double-gasketed segment with convex radial joints – principally to "flex" during earthquakes so that the tunnel remained sealed from gas. A six-month, full scale, laboratory testing was conducted, at the University of Illinois, to evaluate tunnel structural capacity under seismic and ground loadings. Tests of gas leakage through gaskets were also performed. For another level of redundancy against potential gas intrusion, the final liner design also included a slightly oversized tunnel – about 15 cm on the radius, to allow for future placement of HDPE and additional concrete to secure it. As for the existing operating portions of the subway, Eastside Extension design called for continuous gas detection in the operating tunnels and additional emergency ventilation.

2.2.3 *Fault crossing*

During geotechnical explorations for the tunnel in 1996, geologists postulated that an escarpment (approximately 13 m rise in the ground surface over 200 m) actually represented a potentially active buried thrust fault. (A buried thrust fault ruptures at depth, and causes surface deformation (bending) but not an abrupt offset). Extensive geotechnical investigations were carried-out to estimate recurrence intervals and deformations – should a seismic event along this feature occur – and develop design criteria. Neither the tunnel nor the thrust fault are linear features, and the tunnel crossed the fault in three locations. At two locations, Metro designed a more flexible steel liner to accommodate movement.

2.3 *Politics and public perceptions*

Legislation, both federal and local, has impacted Metro System overall, and stopped the Red Line Eastside Extension as it was going to construction.

Overall system delays occurred after the methane explosion investigation in 1985, when the Los Angeles congressional representative amended the subway's funding bill to contain the provisions that tunnels would not be constructed in the methane zone. (A reversal of this legislation – given new technology (for example pressure-face TBMs), was approved 30 years later in September 2006).

Mainly due to some publicized events, including those briefly described in section 2.2.1, along with an economic recession, and the perception of construction cost over-runs, local legislation was passed by Los Angeles County voters in 1998, prohibiting use of local funds (the half-cent sales tax) on subway construction.

After discussions on funding and tunneling safety, Metro conducted a "re-structuring plan" and a "Regional Transit Alternatives Analysis" to evaluate available funding and fixed guideway alternatives to heavy rail for the three transit corridors – including the Eastside. The Alternatives Analysis and environmental process was repeated 1999–2002, with the preferred alternative for final design being light rail technology (LRT), for the most part at-grade, with a 2.7 km of tunnel and two underground stations. The LRT project, The Metro Gold Line Eastside Extensions (MGLEE) would extend an existing LRT, the Metro Gold line, from Union Station to East Los Angeles.

2.3.1 *Final design MGLEE*

The new alignment had some tunneling advantages in that it became aerial or at-grade west of the Los Angeles River and thus avoided highly contaminated areas. Much of the previous tunnel design could be used (pressure-face TBM specifications, double gaskets, etc.) with adjustments for LRT tunnel diameter. On the other hand, the relationship between the tunnel alignment and the escarpment changed, and additional geotechnical investigations were undertaken to evaluate station design, where one station was nearly parallel to the feature.

Final design included assessment of construction contract form and packaging, with consideration of bidder qualifications, possible incentives for performance, and use of the Design-Build (DB) contracting methods. Ultimately, the project was let in three contracts: an underground segment (station excavations and tunnels, by Design Bid Build (DBB)), the at-grade portion (including final station structures), and a bridge. The at-grade/station finish contract was DB. The underground contract was advertised first and separately, but was re-bid months later concurrently with the DB contract. Initial bid to award duration was over 18 months.

2.4 *Right-of-Way, permits and third party items*

The LRT tunnels were aligned more closely with the public streets, however, the street alignments, had some abrupt changes that could not be accommodated with TBM tunneling radii.

This along with some shallower reaches led to design of a grouting program for about 30 structures. Thus, in addition to property acquisition for the station sites, subsurface easement for the tunnels, and construction easements for grouting had to be obtained. Over 30 properties needed to be researched for title and negotiations with property owners etc.

2.5 *Funding*

Transit funds in the United States typically consist of Federal, state, and local money – and much competition between public agencies for available funds. To receive the funds, agencies must comply with various conditions; e.g. completion of Alternatives Analyses and the EIR, which involves a significant public comment element. The MGLEE went through this process twice for the environmental documents, and was finally granted \$490.7 million from the US Department of Transportation, or slightly over half of the estimated \$899 million required for the project. Of this, approximately \$200 million was estimated for the underground (DBB) portion of the project. The construction of both contracts was awarded to a Joint Venture (JV) of Washington Group, Obayashi and Shimmick, with the tunnels subcontracted to Traylor Brothers and Frontier Kemper JV.

2.6 *Construction*

Construction of the MGLEE project began in July 2004 with tunneling beginning in February 2006. At this writing, underground stations have been excavated and tunnel construction is about 70 percent complete. The EPB technology has performed better than expectations, as measured ground settlement has not exceeded 1.0 cm.

3 EAST CENTRAL INTERCEPTOR SEWER

ECIS is an 18.4 km, 4.7 m excavated diameter sewer that increases Los Angeles' wastewater infrastructure capacity. Construction was recently completed, and the new sewer was placed into full operation in August 2004. From any perspective, the project is considered highly successful. There were few construction impacts to the community, costs were within budget, and schedule slippage was managed. Nevertheless, the total duration from concept to completion was over 15 years.

The tunnel alignment runs west from the Los Angeles River east of Downtown, and generally stays under east-west oriented streets until it turns south onto La Cienega Boulevard. Finally, it turns west under the Baldwin Hills and connects to an existing Sewer in Culver City. Figure 3 shows the ECIS with respect to the Metro Rail System.

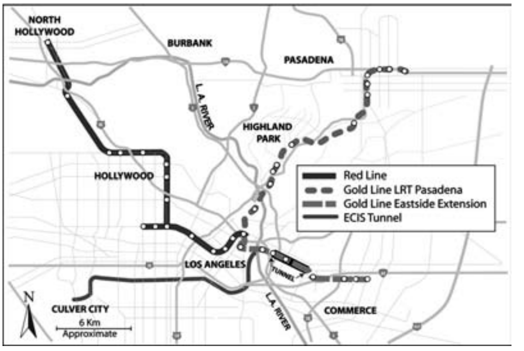

Figure 3. ECIS and existing Metro Rail Systems.

3.1 *Planning and environmental processes*

The City completed various planning studies for ECIS, including an Alternative Alignment analysis and an initial draft environmental impact report in the early 1990's. Opposition to the project from various neighborhood associations – objecting to construction disruption and potential sewer odors – caused the project to be placed on hold in 1995. The initial Environmental Impact Report was never completed.

"El Nino" rainstorms in the 1997–98 winter season caused the city's sewer system to overflow and major sewage spills. The State of California Regional Water Quality Control Board took action and mandated the completion of the ECIS and six other sewer projects within a tight time schedule. Fines were established for exceeding the scheduled completion milestones.

A second EIR addressed concerns raised during the first Draft EIR. For this process, the City emphasized having community leaders endorse the project. There was still considerable community opposition to the project, but the City Council approved the Final EIR in November 1998. (City of Los Angeles, 1998).

The decision to tunnel the entire 18 km as opposed to using cut and cover methods was made during this planning and environmental documentation phase. An outcome was that drop structures had to be designed to divert flows from the existing collector sewers into the new ECIS pipe. The existing collector sewers averaged in depth around 7.6 m, and the invert for the ECIS tunnel averaged about 20 m.

3.2 *Design and technology*

The final design began in 1999 and took approximately 18 months to complete. There were several significant technical challenges for tunneling including minimizing ground settlement. This specific issue drove the City to specify pressure-face (EPB or slurry) TBMs. Discussions among designers and the City's Board of Consultants transpired as open shield technology could still be considered appropriate for much of the tunnel's reach, and cost savings might be realized. Ground was mostly very dense sand with clays and silts, above the water table, and generally considered ideal for the open shield. In addition, given the relatively small diameter, at over 15 m depth in many reaches, meant that predicted settlements would be within the 20 mm allowable presented in EIR. Ultimately, given the Metro experience and the raised public awareness of tunneling risks, pressure-face machines were specified.

3.3 *Bid and award of the tunnel contract*

Given this first use of pressure-face TBMs in Los Angeles, The City implemented a contractor prequalification process to ensure that only experienced tunnel contractors would be allowed to bid.

The project went to bid twice due to issues related to these prequalifications. In the first bid period, advertising began in March 2000. The second bid package advertised in September 2000 and was awarded to Kenny/Shea/Traylor/Frontier-Kemper, JV for $240.4 million in January 2001.

3.4 *Politics and public perception*

As described above for the MGLEE project, during the mid 1990's, there was a high level of public apprehension concerning tunneling in Los Angeles.

During this time, the City had investigated alternative construction methods including open trench construction, which initially appeared more cost-effective and would impose less tunneling risk. The advantages and disadvantages were raised with the community during the Environmental phase. The process resulted in demonstrating that the advantages of tunnel construction greatly outweighed the impacts caused by open trench methods. Public comments during the process also resulted in a change of alignment for the final design.

3.5 *Right-of-Way and protection of property*

Acquisition of shaft sites, laydown areas, temporary easements, and subsurface easements was a critical activity. The intent was to locate the ECIS under public right-of-way, but there were areas along the alignment where the sewer had to cross under private properties. For the 18 km tunnel, over 1,000 real estate transactions were performed including title searches, appraisals, negotiations, purchases, construction easements, and subsurface easements, and starting eminent domain proceedings.

This effort took over 18 months to complete, running in parallel with final design, as the Environmental Process requires that no real estate transactions be made until the Final EIR has been adopted.

3.6 *Construction of ECIS project*

The original construction schedule estimate was 3.5 years. During construction, there were several unforeseen events. These resulted in seeking a contract extension to avoid fines for late completion. The milestone date to place ECIS into service was changed from November 30, 2003 to August 30, 2004. Major unforeseen events impacting schedule included:

– Discovery of a Native American grinding stone
– Re-sequencing work at a work site to allow access to four homes
– Additional precautionary permeation grouting at selected locations
– Complex real estate transactions for acquisition of one shaft site
– Slower tunneling excavation rates

3.7 *Funding*

The project costs were paid through the City's Sewer Construction and Maintenance Fund, and a State of California Revolving Fund that loans to water agencies.

Documentation required by the State for loan eligibility included:

– A Value Engineering study
– Minority/Women/Other Business Enterprise quarterly reports
– The City's water conservation plan
– Draft revenue program
– Project performance certification after one year of operation

3.8 *Changes in the duration of the project*

To offset the delays caused by the unforeseen events, various activities were accelerated. Some of these included:

– Awarding the contract before all right-of-way was acquired: The City assumed a risk of property not being available
– Specifying a fourth TBM
– Acquisition of additional land for a second pipe laying operation
– A 6-day workweek with two 10-hour shifts
– Change to a prefabricated pipe at one drop structure – in lieu of a concrete cast-in-place option

4 CONCLUSIONS

For tunnel projects in urban areas to be built in a reasonable time frame, planners and designers must expect and plan mitigations for interruptions in the project schedule due to environmental issues, funding, and the bid process as well as develop technical solutions for design. From these examples alone, we see that major changes are not uncommon as a result of the Environmental Studies and public comment.

Planning, design and construction of major infrastructure projects in urban environments requires many years and is costly. In planning, the agency must also use demand projections that look 50 to 100 years into the future to maximize the value to the public. The examples presented here are a clear indication of the long-term effort associated with providing a very much needed improvements to the communities they serve.

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Geological and geotechnical conditions in underground structures design of Rome subway: the B-Line extension

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ABSTRACT: Underground works, in complex urban areas, can present prominent levels of risk connected with various ambits: low overburden, buildings facing the line, interference with acquifers, soil of poor geotechnical characteristics, presence of man-made cavities. The knowledge of the geological – technical context is the first and inevitable stage of study, for the development of an underground construction project, by which it is possible to achieve a high level of sensibility about the underground space and to propose mitigation measures for the risks associated, both during the construction phase and during exploitation. The case of Rome subway B-line extension is presented, where a rigorous procedure, involving geotechnical investigations and consecutive phases of study, has been applied both for the preliminary design and for the construction phase (PAT – Protocol for Advancement of Tunnel excavation). The illustrated approach has been consolidated from the experience of many similar completed projects, and has become a standard practice.

1 THE WORK AND THE TRANSPORTATION SYSTEM OF ROME

1.1 *The "Mobility System" of Rome*

The transportation planning of a city like Rome presents a high level of complexity and requires the choice of strict planning tools.

The "Mobility System", drawn up and approved by the local administration, is the reference programmatic document, in which the different existing transportation networks, the actions for their development and updating find a rational placement and harmonization.

According to the "Mobility System", the subway network proves to be a necessary means of transport, which is able to connect, in a rather short time, the highly populated areas of the suburbs to the city center. This aim is being further developed, as shown in the ambitious enlargement plan (Fig. 1).

1.2 *The subway B-line extension*

The B-line extension project coherently fits in the above described mobility scenario, enabling the old line to settle close to the existing ring-road highway (Grande Raccordo Anulare, GRA) and thus representing an important nodal exchange (Fig. 2).

The B-line extension includes 3 km of new line, 3 intermediate stations and a new depot. In this way, the San Basilio, Torraccia and the new Casal Monastero districts will be served for the first time, with the view of a contextual requalification of their urban and social context and also installing new services.

From a technical point of view, the new metropolitan section will be realized entirely underground, with excavation between retaining structures and below the cover slab (top-down method). This will allow the surface conditions to be restored quickly, limiting the disturbance to the public to a relatively short time and ensuring the separation of underground works from ordinary daily activities.

1.3 *The assignment*

The engineering services, necessary to the preliminary and definitive design of B-line extension, has been assigned by the local administration (Comune di Roma) to the group of companies composed of C. Lotti & Associati, Geodata, Siteco and A2G.

Figure 1. The subway network enlargement plan and the location of the B-line extension.

Figure 2. The B-line extension design in the Casal Monastero area.

2 TERRITORIAL CONTEXT, POSSIBLE DIFFICULTIES AND BASIC PHILOSOPHY

The project of the new subway section had to face many difficulties, connected to the insertion of the works in the already complex territorial context of reference, due to specific technical uncertainties, which were pointed out since the very first planning stages, as being associated with an initial level of high risk.

More specifically, the major difficulties were proved to be directly or indirectly connected to the geological-technical context that consequently assume high priority on the design agenda, even before specific studies and geotechnical investigation took place.

Figure 3. Collapse of a cavity in Rome (Via Bricci).

The most important uncertainties, directly connected to geological-technical context, are the following:

- type of soil intercepted by the excavation, with highly variable level of cementation and poor geotechnical characteristics;
- presence of an unconfined aquifer at the tunnel level and possibly other deeper aquifers, even in pressure;
- verified presence of man-made cavities in the site, single or complex ones, also of big size (Fig. 3).

The most significant aspect indirectly connected to the geological-geotechnical context is the presence of many buildings along or facing the line of the work.

It is therefore clear that the full knowledge of the geological-technical context is an indispensable prerequisite for the design. This is not only for normative reasons or design completeness (as often required today), but also for the management and mitigation of the risks during construction.

This is especially effective if performed since the very first steps of a project, that is since the Preliminary Design, when specific studies and geotechnical exploration can be successfully used to reduce uncertainties while work's structural designing can be directed at best.

This process will ensure the most effective definition of:

- risks connected to the construction, with reference to the design;
- functional level provided by the line;
- resulting economical and financial schedule.

The value of this approach can further increase once extended to the construction phase by a constant monitoring of the excavation parameters and by the analysis of the work's actual conditions (PAT Method: Protocol for Advancement of Tunnel excavation, Grasso et al. 2002a,b). These are strategic activities for the risk management and its mitigation, as they ensure a real-time comparison between the design scenario and that encountered during construction.

The B-line extension of Rome subway, designed with a deep awareness of the above described philosophy, well illustrates the Geodata's methodological approach.

3 THE METHODOLOGICAL APPROACH

The methodological approach followed for the design of the B-line extension is based on a well defined succession of phases; each phase was established on the results of the previous one. In this way a sequential "cascade" process was introduced, by which the activities of every single phase proved to be efficiently rationalised (in terms of quantity, typology and costs) as systematically addressed by the issues of the prior phase (Fig. 4).

The process, by favouring a constant comparison with the Owner, benefited from the contributions of the whole project management team, thus permitting a better definition and management of risks.

In the following pages the method outline will be described and subsequently account will be given of the level of knowledge that has been reached by its application.

3.1 *Consolidation of basic information and preliminary geological-geotechnical model*

The first phase of the process is the collection of all the information and data concerning the geological-geotechnical context, that were available since the Preliminary Design started off. They are: general bibliography, available stratigraphic evidences, technical studies and previous design documentation (in particular, Feasibility Design). This first phase proved to be a strategic one, because it led to an immediate awareness of the context in a very short time.

Technical and reconnaissance surveys were also performed, so that the gathered information could be set in its proper context and the knowledge of the "geo" scenario could be further investigated in a rational way, minimizing the impact on the urban activities and reducing time-cost for the geotechnical exploration.

A platform of GIS (Geographic Information System) was prepared for ground data filing, consultation and real-time updating.

It should be noted that in complex areas, such as Rome, with buried channels and erosion forms, complex lateral eterophies between alluvial and volcanic sediments, etc. reliance must be placed on engineering geological interpretation of available information, prediction on the basis of known geological relationships and careful interpolation and extrapolation of data.

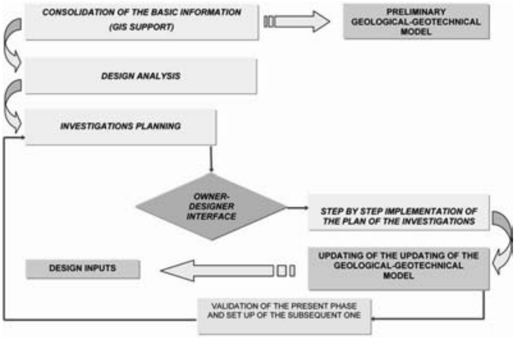

Figure 4. Methodological process flow-chart.

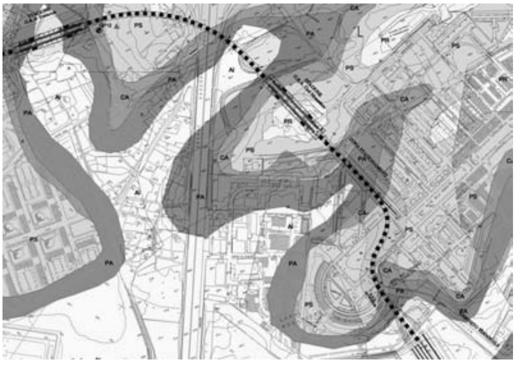

Figure 5. Extract of the Geological Map: actual alluvial deposit (hatched bright zone) and volcanic sequence (hatched grey-scale zone).

For this reason inside the investigations for the design of Line B the value of the geological surveying phase and the presence in the geological and geotechnical design team of specialist in Structural Geology and Sedimentology were emphasized: also in urban areas, with limited rock and soil outcrops the "geological survey" (improved in this environment with trenches, shafts and other indirect data) made it possible a correct interpretation of the results obtained from the deep investigations. At the end of the first phase a preliminary geological-technical model was defined, substantially represented by a geological map (Fig. 5) and by the corresponding profile along the axis of the work.

3.2 *The project and the plan of site investigation*

After the "geo" context was defined, even if preliminary, the design phase started off; its development took into account the potential risks that had emerged in the previous stage. The hypothesis of the project was since the beginning aimed at reducing risks of the construction phase.

In parallel, once the uncertainty margins of the "geo" context were determined, a specific investigation plan was developed and validated by the Owner; it included different steps: geophysical (Stage I) and geotechnical ones (stages II and III).

3.3 *Update of the geological-geotechnical model*

Once the investigation steps I and II were performed, a first update of the geological-technical model followed, based on which significant design indications to manage risks were given.

4 PRELIMINARY GEOLOGICAL-GEOTECHNICAL MODEL

In the following paragraphs the results of consolidation of the basic information will be showed, with reference to the geological-technical investigations. The analysis of every ambit of study ended up with the characterization of the difficulties connected to the construction, on which the subsequent investigation steps was planned.

4.1 *The geological scenario*

The area of interest is located inside the Thyrrhenian margin of Apennines. Its complex geomorphic, stratigraphic and structural elements can be traced back to the geodynamic evolutionary characters of the Apennines chain of the late Pliocenic-Quaternary period (Ventriglia, 2000).

Above the oldest carbonate levels, not relevant for the design, terrigenous sandy-clayey sediments of Pliocene can systematically be found (Argille di Monte Vaticano, Marne Vaticane, Argille azzurre). These units act as substrate of reference for the subsequent elements. Over them, pre-volcanic alluvial deposits occur (Unità di Santa Cecilia), consisting of gravel, clay and sand, organized in complex stratigraphic relationship.

The pyroclastic sequence that follows is of a later age and is linked to the past explosive volcanic activities, which are characterized by old altered tuffs (from Unità di Tor de Cenci to Successione di Sacrofano), pozzolane (Pozzolane Rosse) and lithoid tuff (Tufo Lionato). Finally one can find the actual alluvial sediment and the more recent man-made filling, often remarkably thick.

Considering the complexity of the stratigraphic features, a geophysical and geotechnical investigation was necessary to reorganize the geological elements concerning the project.

4.2 *Geomorphology and underground cavities*

The area is characterized by the presence of river Aniene's right tributaries, which flow in relatively deep flat valleys, whose sides have often been changed by extraction activities. Old or current instability phenomena are not visible, neither along vertical caves faces, because of the good technical characteristics of the volcanic soils.

Figure 6. Cavities found along the line with buildings above (Pietralata area).

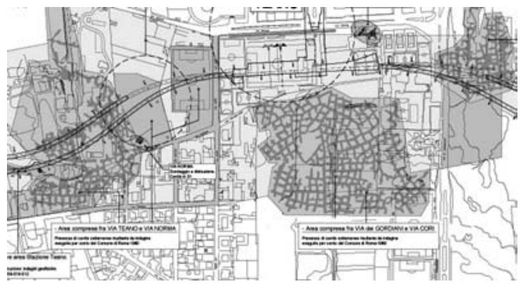

Figure 7. Map of cavities surveyed (hatched grey zone) and risk of being intercepted.

In the concerned area underground cavities can occur; they can be caused by the extraction of lithoid tuff (Tufo Lionato) and Pozzolane Rosse, especially in the areas of S. Basilio and Casal Monastero, where the great thickness of the volcanic units favoured the extractions (Fig. 6).

Single and isolated cavities can also be found everywhere, especially inside the pyroclastic sequence; they were generally excavated to function as a well or water storage tank.

Since the beginning, the identification and mapping of man-made cavities proved to be a necessary step, both for technical (difficult realization of support works and excavations) and for safety reasons (buildings near the excavation areas, underground utilities networks, etc.). This activity was performed through surveys, within the planned site investigations, systematically integrated with the GIS platform (Fig. 7).

4.3 *Geotechnical and hydrogeological issues*

The involved lithologies show a high degree of heterogeneity in terms of mechanical characteristics and cementation, in particular:

- pre-volcanic alluvial sediments show prevalent silty granulometry, low density and high water content; occasionally cemented levels. Generally they present poor mechanical characteristics (both resistance and deformability);
- pyroclastic sequence shows a stratified aspect, with alternation of altered (incoherent, mechanically poor) and competent (lithoid tuff) levels;
- actual alluvial deposits show clayey-silty granulometry, good plasticity and high deformability;
- man-made fills present a highly variable granulometry, with incoherent behaviour.

Some potential, major, construction consequences derive from this complexity: for example, the presence of lithoid levels makes difficult the excavation with traditional tools, as well as the possible escape of bentonite-slurry used for the excavation of retaining walls.

A detailed geotechnical investigation was therefore necessary, in order to examine every encountered lythology, both in situ and in laboratory.

The potential hydrogeological problems can be related to underground water at the level of the design (interference with excavations and supporting works, dam effect, hydraulic overpressure); possible deterioration of the setting by underground cavities.

To manage risks at best, all the drillings were provided with stand-pipe and Casagrande type piezometers, for a continuous monitoring of piezometric levels.

5 THE SITE INVESTIGATIONS

5.1 *Step I – The geophysical investigations*

The first step of investigation was performed by combining geophysical surveys of georadar and geoelectrical methods, developed along work's longitudinal axis; the results were reciprocally merged for a better resolution. Site calibration by the selected methods was conducted, accomplished by two field-tests focused on searching for man-made cavity.

Geoelectrical tomography (multi-electrod with spacing <3 m) highlighted resistivity values of 50–200 Ωm for the top layers (pyroclastic sequence) and lower values for the deeper layers, 5–50 Ωm (pre-volcanic alluvial deposit, completely saturated). No significant cavity was detected.

Two resistivity anomalies were identified in the pre-volcanic alluvial layer (400–600 Ωm): subsequently tested with boreholes, they turned out to be portions of a very loose soil, partially washed out by groundwater flow (Fig. 8).

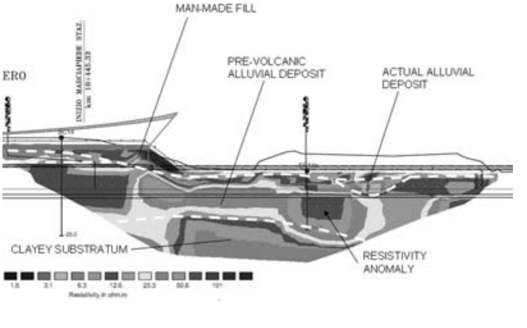

Figure 8. Geophysical section with resistivity anomaly identified, subsequently investigated with test boring.

A multi-array georadar survey was then carried out, for a multiple frequency scanning of the underground context. At first, frequencies at 100–200–600 MHz were adopted, with a lower penetration but higher resolution. Later, an ultra-low frequency antenna was used (25 MHz) for not build-up area only, where no disturbance-factors should be present for the radio signal. Its maximum penetration reached 10 m.

On the whole, the georadar survey has highlighted a complex network of underground utilities as well as the stratification of the upper portion of the pyroclastic sequence.

5.2 *Step II – Geotechnical investigaion*

According to the findings of the previous step, a first geotechnical investigation was conducted, with core borings and laboratory tests: geophysical anomalies was directly explored as well as the most interesting areas.

In total, 27 test boreholes were drilled, all of which provided with piezometers (open stand-pipe or Casagrande stand-pipe, occasionally coupled in the same borehole, for simultaneous logs of different aquifers).

SPTs, Lefranc, Lugeon and pressiometer tests were systematically performed in the boreholes, for a better definition of the geological and hydrogeological context.

5 piezocone tests (CPTUs) were carried out on the alluvial deposits.

Ultimately, for archaeological reasons, 9 additional borings were drilled around the presumed-ancient building of Casal Monastero.

6 UPDATING OF THE GEOLOGICAL-TECHNICAL MODEL OF REFERENCE

At the end of the process the preliminary geological-technical context (Fig. 9) was updated (Fig. 10).

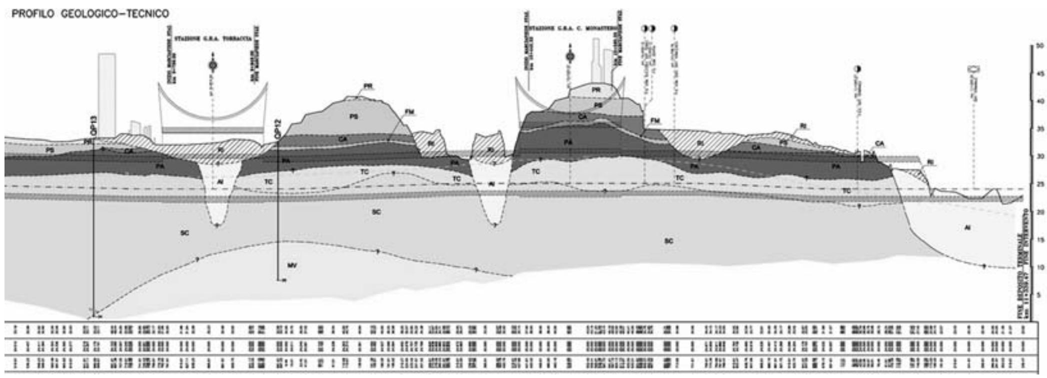

Figure 9. Profile of the preliminary geological-technical model.

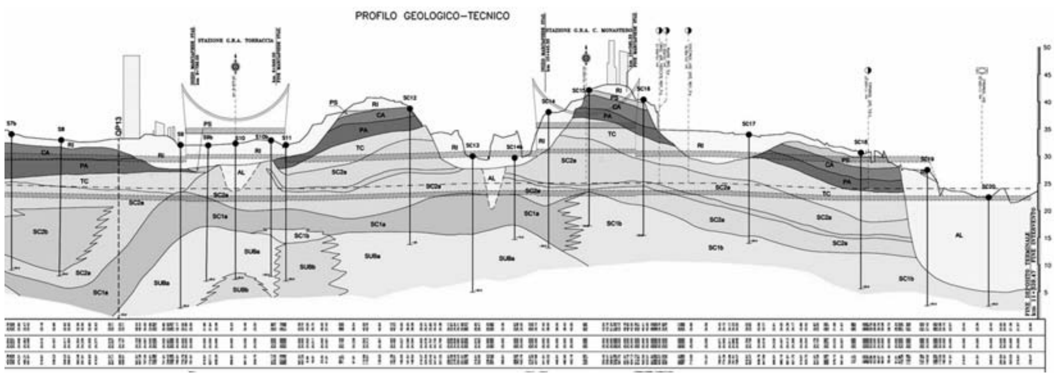

Figure 10. Profile of the updated geological-technical model.

The higher level of details permitted a considerable reduction of the uncertainty margin as well as risks associated with the construction of the subway extension.

In parallel, the GIS archive was kept up to date, with a final reconstruction of the geological-technical model, through 3D rendering.

7 CONCLUSIONS

The proposed method has shown how a rigorous protocol of study, rationally developed, could permit a timely identification of the critical aspects for the design, the further studies and management of these critical aspects in the design process, till the selection of the most appropriate typological solutions for minimizing the connected risks. This has been implemented step by step, with an easy-to-follow-again methodology, both by the Owner and by the Designer.

The whole process was followed by the continuous development of the geological-geotechnical model in GIS environment. This activity will proceed in the further design stages and will be completely accomplished in the construction phase, with the implementation of the PAT protocol that will en-sure the management of risks and any unforeseen situations through the most efficient counter-measures, predefined in the design stage.

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Ontologies, thesauruses and information systems for tunnels

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ABSTRACT: Usefulness of glossaries has not to be proven in the tunnel conception environment. However, glossaries are limited to simple lexical information; semantical information could be interesting to add for multiple uses.

Ontologies allow this addition, along with reuse and easy implementation in a computer system. This paper aims at showing what is an ontology and how it could be built for the tunnelling community. By the tunnelling community, too, for one of the main backdrafts of "classical" information systems ontologies are their high level of technicity. Some ways of avoiding this problem are proposed.

As an illustration, ontology-driven projects are presented with their application and first results. One of the most promising of these projects, more precisely described, combines rule-based knowledge representation and ontology to allow expert-supervised and semi-automatic compliance of a tunnel project to a set of norms or recommendations.

1 INTRODUCTION

This paper deals with some considerations about knowledge management for tunnels. It features at first some elements about terminological knowledge and existing tools, as it is true that "knowledge representation begins with words" (Sowa, 2001), then it unfolds some considerations about the adaptation to other tools (ontologies) for the tunneling community.

What is presented here is a point of view built from the early experience given by various existing or under development projects of knowledge representation and management for the tunneling community. These projects are intended to give tunneling experts and specialists assistance during the course of a tunnel-related work (types of said works varying with individual projects). They are mainly computer systems conceived to be complementary with existing computer tools (which deal mainly with graphics and mechanical calculus).

2 A VARIETY OF TOOLS

When it comes to collecting words, one think to indexes (a collection of words) or glossaries (an index, associated with definitions). But there are more tools than just indexes and glossaries. Let's have a quick look at them:

– a **taxonomy** is some kind of index, organised along some hierarchical guidelines. Taxonomies are often built along the specialisation/generalisation guideline (specialisation: from rock to metamorphic rock to granite; generalisation: the other way – from granite to rock). This induces some limited semantics, even if this can be constrained to the terminological field with the hyponymy/hyperonymy relations (narrower/broader term). Taxonomies can vary, however: they have often one type of relation only, but may also feature other relations – meronymy (part-of relations) is often chosen. In our example, Rock has a narrower term, Granite, which has some components (Quartz, Mica, Feldspar).
– taxonomy introduces some semantics, but do not take well into account the alternative uses for some words (polysemia). This is where the **thesaurus** intervenes, which features, as in a glossary, definitions, and as in a taxonomy, hierarchical relations; but also similarity relations ("see also", "related terms") – which can be used for figuring polysemia, but for other information retrieval goals as well (e.g. finding Music from Rock).
– thesaurus deal not only with terminology (defined here as the collection of terms and their meaning), but dabble also with semantics (defined here as the relationships between various meaning of one word – or group of words). The step beyond is **ontology**. Originally borrowed from philosophy (where the term stands for "the study of what is in being"), ontology has been used in the information systems field from the early nineties. To most people, ontologies are gigantic thesauruses, and indeed they are, but they are more systematic than thesauruses: they depict the knowledge domain,

while the thesaurus is limited to information retrieval.

The next part of this paper expands this last point and describes ontologies, in their diversity and in their uses.

3 ELEMENTS ABOUT ONTOLOGIES

A quick explanation of what is an ontology could be its most famous definition, given by Gruber (1993):

"An ontology is the specification of a conceptualization".

In other words, and at greater length, ontologies rise from another problematic entirely than thesauruses: the matter with ontologies is: "how to describe synthetically the knowledge about things?" The second thought behind this one being, of course: "and use this knowledge for some computer-assisted task". So, conceptualization is a set of objects and constraints between these objects which represent what is known about the field these objects belong to. But, as the whole thing is about knowledge, and knowledge does not belong to the physical world, but is rather applied to it for understanding, these objects are concepts. And as concepts are fundamentally ideas, something of the mind, they are expressed by words. (There is need here for a lot more theoretical explanation, but this should be quite off-topic).

It explains, then, why thesauruses and ontologies look strangely familiar: they are both made of words and relations; it explains also why they are essentially different.

But then, there is the specification part.

3.1 *Models and concepts*

Building a structure for concepts, terminological considerations being irrelevant, is another matter entirely than building some reference structure for general ideas (although this is a quite arduous task, either). This is where models come.

Models are, simply put, some "strategy of irrelevance", which means that it is the result of a choice of objects and relations relevant in a given context, and the deliberate exclusion of every other.

This way, models are everywhere. The "simple" glossary begins with a simple model, for it has a theme, it concentrates on a set of words relevant for a domain; and our geotechnical thesaurus can be rid of the junction between Rock and Music, and Music itself – as Music is irrelevant in a geotechnical context. This is called a "domain", a field of knowledge. The ontologies we discuss here are domain ontologies (some ontologies are deemed global, but their use and "tuning" is very uneasy).

Conceptualization, then, is also a modeling, and as such do not differ greatly from the simple models except that it is formally specified (although this is sometimes discussed). Which means that a simple, natural-language definition of a concept is not sufficient for this concept to be formal: it needs more depth, a definition relatively to other concepts, and a sign (say, a word) exclusively attached to it.

As a result, the concept appears as a category: the set of all things in being that matches its definition. This definition being itself a set of attributes, which are, in turn, concepts of their own right elsewhere in the ontology. This is an old logical approach, but it works quite well with the object-oriented languages of computer science and their logical counterparts (currently, the OWL language is becoming popular for building web-based ontologies).

What does all this imply for tunneling?

3.2 *An ontology of tunnelling*

From what have been said about ontologies, we can conclude rightly that an ontology is essentially context-dependent, or at least model-dependent.

Research has recently evolved towards a multi-modeling conception of ontologies, which means that multiple models can co-exist in an ontology. This is done, more or less, with what can be called "point of view" approaches: for example, one point of view sees the road as a surface where traffic evolves according various parameters, another sees the same road as a concrete layer upon a compacted patch of soil, in which water infiltration has to be minimal. For the same concept "road", attributes and distribution of these attributes is different – modeling is different.

This could be an interesting way of building ontologies in such a diversified field as tunnels-related activities (even considering narrower ontologies such as tunnel construction, tunnel maintenance, safety in tunnels, etc…).

However, much of these point of view approaches still depend on a broader model – extremely arduous and painstaking to obtain in the case of geotechnics, as it needs a broad consensus and a formal definition of every item (which, due to the probabilistic nature of everything soil-related, is quite impossible to obtain).

It should therefore be interesting to build a specific model for each tunnel case, from a knowledge base or another kind of existing data, but without relying on a previous, global model.

That is the point of the RAMCESH project and affiliate projects, led by the SOLEM France society.

3.3 *Induction rather than deduction*

More in line with case-based reasoning (CBR), a new approach for building ontologies is considered: a global formal model is not the starting point, but the end of ontology building, resulting from the combination of various specific, rather incomplete, models.

These incomplete models (“pieces” of models) are fairly common in the geotechnical literature: observations of some precise material behavior, general consideration about a technical implementation, etc…

These pieces of models, represented in some proper formalism (which should be quite simple and manageable by other than computer scientists), are then combined according to some project considerations in a broader piece of model, and so on until a coherent and rather exhaustive model is attained.

Of course, this type of combinational model has the risk of being not so exhaustive, for every piece of the ideal project model does not exist in the literature, or rather in the knowledge base built from the literature.

But remember: this is a computer-assisted system that we discuss here, not a computer-based tunnel project builder. This approach needs domain specialists and experts for more than supervision, but also for exploitation of the results. And better for them to “fill the blanks” in a model than to build this model from scratch.

In the RAMCESH project, those pieces of models are named “knowledge grains” and use informal concepts; the concepts gaining increased formality by being combined with other grains.

4 ONTOLOGY: TUNNEL-RELATED USES

The general use for ontologies is communication (Uschold & Gruninger, 1996): communication between people, systems and/or organizations.

For example, ontologies are at the center of what is called the Semantic Web, which aim is to allow “intelligent research” among the Internet – or more specialized networks. The ontologies used or planned in the tunneling community have this same aim.

The CONFEC7 project, for example, is aimed at building an ontology from the normative knowledge contained in the Eurocode 7 (the new European norm about geotechnical calculus); from this point on, every geotechnical project (expressed in an appropriate formalism) can be easily reviewed according to this knowledge, and estimated compliant or not to the norm.

UPTUN is an European Community project focused on the upgrading of safety in tunnels. One of its parts uses an ontology, which describes the various security systems and the way they interact. According to some user-provided data (through various forms describing intended safety modifications), it describes the way safety evolves in a tunnel.

Aforementioned RAMCESH project (from which CONFEC7 project is derived) focus is to give specialists the opportunity to develop their tunneling projects

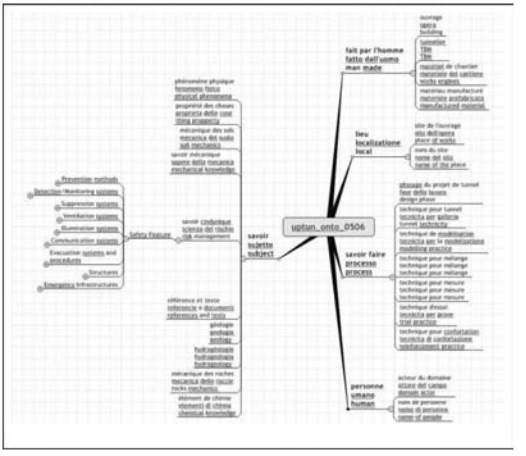

Figure 1. An overview of top-level UPTUN-Onto concetualization.

according to some literature-supported specific theories, then to automatically compare the way they have conceived the project with other projects, stored in memory or conceived by other specialists.

Other uses considered in these projects and others include tendering (roughly, estimating prices and delays from the description of a tunneling project) or day-to-day survey of field works and panel of optimized choices according to the results. More documents-oriented applications are possible, too, as the automated indexation of documents (this is a direct semantic web application), but they only indirectly imply tunnels.

One subject of difficulty when building such tunneling ontologies is that it is quite time-consuming for experts to build ontologies. And as tunnel experts are required to build tunnels and not ontologies, this is a major problem. To address it, the general choice made in the tunnels-related ontologies projects mentioned before has been to develop network solutions and sequential procedures to allow various experts to implement the system easily, and not necessarily during long working sessions.

This choice is also related with some conclusions drawn from the expert systems experience for tunneling activities of the eighties-nineties: the implication of the domain experts in the implementation of such knowledge-based systems must be maximal (Magnan, 1992). Formulation and, foremost, use of knowledge must be an expert domain prerogative: the reasoning process of such systems must be expert-driven, not computer-driven (hence, the computer-assisted bias mentioned in chapter 3.3).

This leads to such approaches as UPTUN-Onto or the RAMCESH ontology construction system, oriented to increased user-friendliness, for the ontology user and builder is the tunnel expert.

One more important point about ontologies when oriented toward such a large and international community is that they have an edge regarding multilingual possibilities. Being conceptually modeled, ontologies are virtually language-independent, i.e. concepts and their relations are the same, only their designation change. This point is discussed in the case of cultural facts, but for the main part, tunneling ontology (featuring physical concepts) should be culturally independent.

Computer-wise, ontologies are fairly portable, as they rely on current and easily accessible technologies, mainly XML and XML-related languages and formalisms (RDF, RDFS, . . .). Therefore, it should not be a technical burden to any organization's computer division to deploy such systems.

5 CONCLUSION

The few applied experiments – that we know of – of ontology development for tunneling uses have shown some merits and drawbacks. Merits concern the very nature of ontologies: terminological and conceptual models in a same portable system, high adaptability to network and distributed contributions, modularity in modeling process. Drawbacks are often directed to the computer-system dimension of ontologies: languages and logical approaches far too uncommon for the tunneling engineer and high degree of formalism required where it is not always possible.

However, these drawbacks can be dealt with, as illustrated in this paper with some current projects and theories.

In fine, ontologies are quite adaptable to the tunneling environment and community; regarding the past trials of glossaries and indexes for the domain, it would certainly be profitable to build such reusable models, for both terminological and conceptual motives.

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Building the underground infrastructure of Lausanne's new m2 metro line

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ABSTRACT: A new subway line is under construction in the city of Lausanne, on the shores of Lake Geneva in Western Switzerland. Called "*projet m2*", it is designed for an annual capacity of 25 millions passengers and is scheduled to open in late 2008. It will offer a new "urban elevator" to a city characterized by its steep slopes. The trains of the new fully automatic "driverless" transportation system will feature rubber tires allowing them to climb slopes averaging 6% and reaching up to 12%. It will be the first automatic metro to cope with such steep gradients. About half of the 450 mios Euro project budget will be spent on the 6 km long new infrastructure. It includes approximately 2.9 km of new tunnels, 2 km of new cut-and-cover tunnels, 0.5 km of old tunnels to be renovated and 14 stations. The construction of six separate tunnels in parallel was required to respect the very tight construction schedule and to respond to the varying geological conditions. The engineers and contractors face difficult technical and logistical challenges as these tunnels are excavated at low depth in a dense urban environment. This article will give an overview of the complexity of the project, including different excavation techniques, as well as the point of view of the Client regarding its role in the project managing process.

1 SUSTAINABLE DEVELOPMENT

A new subway line of Switzerland is being built in Lausanne, Switzerland. With a population of about 250'000, Lausanne is a relatively small city for a fully automatic subway system. The ambitious project, named *projet m2* aims at boosting public transportation in a city characterized by a difficult topography and a growing traffic problem. The project received broad public support in November 2002, when the local state population accepted the proposed project funding (overall budget of 450 mio €). Construction work started in spring 2004 and commercial service is scheduled for the second half of 2008. Key project data are presented in table 1.

Covering a distance of approx. 6 km (fig. 3), the m2 line will link the Geneva Lake shore at 373 m.a.s.l. with the suburb of Eplinges at 711 m.a.s.l. (i.e. 338 m higher, which explains the origin of the "urban elevator" expression). Fourteen stations will be distributed along the alignment.

6 km in 20 minutes
1 train every 3 minutes in the central portion
6,600 persons per hour
25 millions persons per year
37,000 jobs at less than 300 m from any metro station
6,000 cars "removed" from the city center

Figure 1. *projet m2* performances.

2 TUNNELLING CHALLENGES

From the "tunneling" point of view, the *projet m*^{2} has been characterized by some challenges requiring the engineers and contractors best performance.

Although most of the m^2 tunnels are at a "standard" low depth of 15–25 m in a relatively good rock (the swiss "molasse", composed by an alternance of sandstone and marls), some segments are very shallow and, sometimes, in weak water saturated soils (sands and gravels).

The most interesting examples are:

– Saint-Laurent tunnel: excavation beneath the traditional masonry bridge "Grand-Pont" (fig. 2).
– Langallerie tunnel: excavation through sands, gravels and silts, 3–4 m under the foundations of Rue Langallerie buildings (fig. 4).
– Viret tunnel: excavation at less than 1m beneath the foundations of the Rue Madeleine buildings (fig. 5).
– Bugnon tunnel: excavation through sands, gravels and silts, less then 1 m beneath an existing tunnel connecting two buildings of a large hospital.

Two others objects are worth mentioning.

The first one is the construction of a new bridge (Pont St-Martin) through the piles and abutments of an existing century old masonry and steel bridge (Pont Bessières, fig. 6).

Due to topography of the city, a bridge was necessary to connect two areas separated by a deep valley.

Figure 2. Saint-Laurent tunnel. Note the passage under the 19th century “Grand-Pont” masonry bridge.

Instead of creating a completely new bridge that would have difficulty to integrate in the urban environment, the designers chose boldly to place the new bridge just below an existing one (fig. 6). The technique adopted to excavate through the piles of the existing bridge (with no interruption of bridge traffic) is a mix of structural engineering and underground technology consisting in:

1. reinforcement of the piles by longitudinal and transversal pre-stressed cables and passive anchors in order to bring confinement to the remaining pile material around the void to be created;
2. umbrella forepoling to reinforce and protect the crown of the void to be created;
3. excavation of the masonry by hydraulic hammer through;
4. construction of a reinforced concrete frame.

Another interesting object is the retrofit of the existing LO tunnel. This ancient masonry tunnel was built in the 19th century for Europe’s first cablecar which linked the Geneva Lake with the city center. In order to adapt this tunnel to the new metro alignment, the foundation of the tunnel needed to be lowered 5 m with respect to the original level. Obviously, this had to be done without damaging the tunnel structure and the above-standing buildings.

The solution chosen by the engineers is (fig. 7):

1. execution of a triple curtain of sub-vertical jet-grouting columns on both sides of the tunnel to provide vertical underpinning of the foundations;
2. first 2 m high excavation;
3. execution of 45° inclined jet-grouting reinforced “anchor-columns”, in order to improve the adjacent ground and to provide horizontal support;
4. repetition of steps 2 and 3 down to the bottom;
5. casting of the new foundation and walls and installation of horizontal metallic props at mid-height.

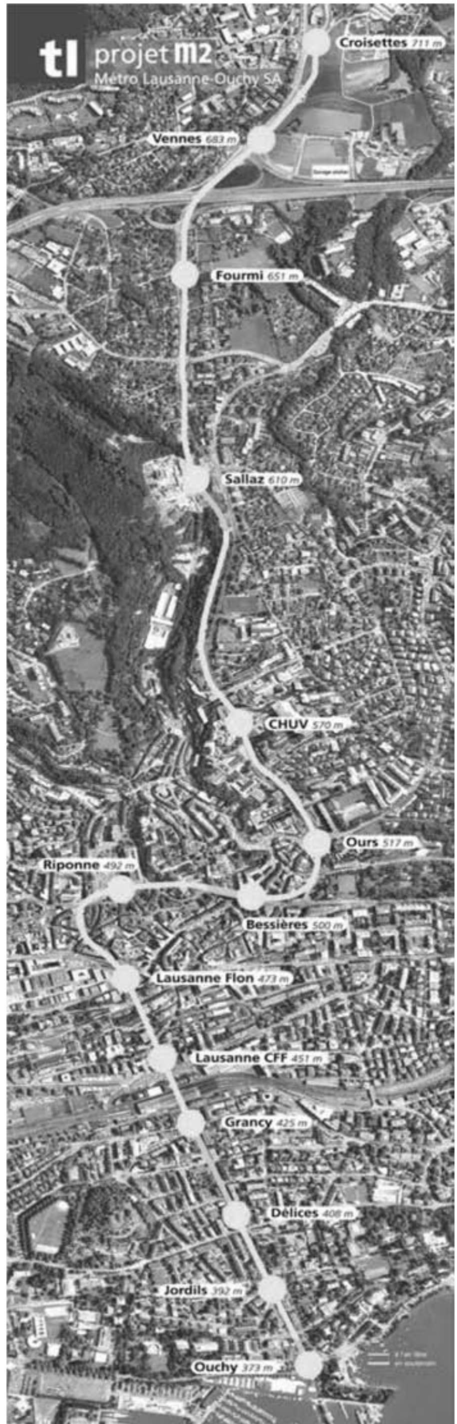

Figure 3. *projet m2*.

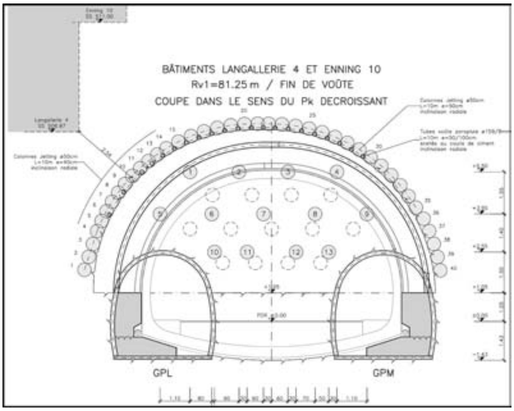

Figure 4. Transversal section of Langallerie tunnel. Note the jet-grouting columns, the side drifts and the proximity to buildings foundations.

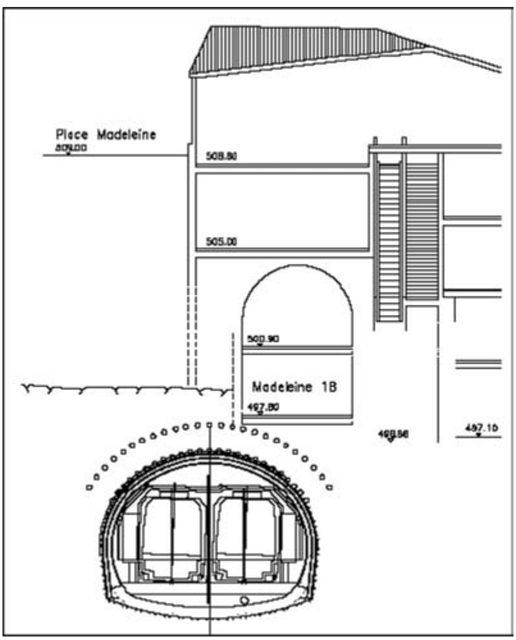

Figure 5. Transversal section of Viret tunnel. Note the umbrella forepoling and the proximity to building foundations.

Different consolidation techniques were used in order to ensure structural safety and settlement control during the construction of tunnels. They include:

– Umbrella forepoling for tunnel crown pre-reinforcement;
– Fiberglass nailing of the tunnel front;
– Jet-grouting, both subvertical and subhorizontal;

Figure 6. Construction of the Pont St-Martin below the Pont Bessières.

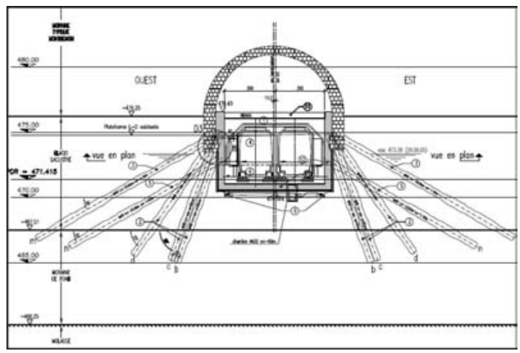

Figure 7. Transversal section of the LO tunnel. Note the existing masonry structure and the jet-grouting columns for underpinning and consolidation.

– Cement grout injections by means of "tubes a manchettes" (non-return valves fitted tubes) for ground improvement.

2.1 *The Saint-Laurent tunnel accident*

The project was unfortunately marred by a serious construction accident.

Late afternoon on February 22, 2005, the front of the Saint-Laurent tunnel collapsed. A large volume (approx. 1500 m^3) of mud flowed into the tunnel, when an unexpected pocket of water saturated sands and gravels was encountered, thus creating a sinkhole that reached the surface 15 m above.

The sinkhole formed under a main square of Lausanne's downtown historic shopping district (Place St-Laurent) and partially under the foundations of a multi-story building (fig. 8). Thanks to Santa Barbara nobody was hurt and the accident only caused material damage.

Figure 8. View of Place Saint-Laurent after the Saint-Laurent tunnel accident. Note the large sinkhole, which extends partially under the building.

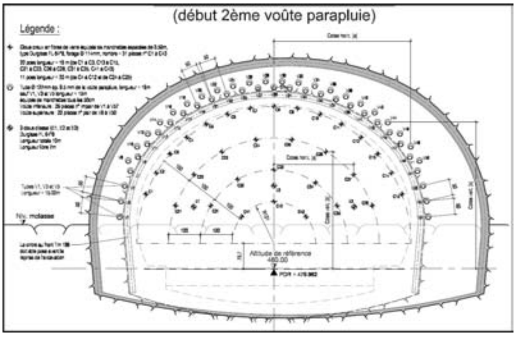

Figure 9. Consolidation measures throughout the critical zone of the St-Laurent tunnel. Note the large number of boreholes for umbrella and injections.

At the moment of writing this article, two inquiries are on-going to determine the causes and responsibilities of the accident, thus no comments on this subject can be made.

The response to the accident required prompt and decisive action by all the parties involved (Besides the engineer and contractor, this includes the Owner and the local political authorities and administrations).

Within two days, countermeasures were taken to stabilize the situation and avoid the progression of the sinkhole which threatened to cause the collapse of adjacent buildings. These measures include:

- Stabilization of the sinkhole walls with shotcrete.
- Further stabilization of the sinkhole with sprayed liquid nitrogen applied in order to freeze the soil at the bottom of the sinkhole and reduce the erosion induced by water circulation.
- Creation of a wall of piles bored from the surface into the tunnel. This wall acted as a “plug” by blocking the flow of material from the sinkhole into the tunnel.
- Filling of the sinkhole with porous material.

Ten days after the accident, the sinkhole was completely filled and the crisis was over. Further measures were taken in the following weeks and months in order to finish the tunnel on time in spite of the accident. Despite its strong emotional impact, the accident did not stop or delay the construction project.

(In retrospect, it is noteworthy for the profession to note that the successful management of the crisis contributed very significantly to reduce the damage to the credibility and image of the *projet m2* resulting from the accident).

After the accident, it was determined that a 35 m long zone of soft and partially disturbed soil between the collapsed tunnel front and a molasse rock formation required special excavation techniques. It was decided to open a new excavation front from the northern adit in order to excavate this critical zone “from the rear”.

The following measures were implemented for the excavation of this critical passage (fig. 9):

- Crown pre-reinforcement by means of a double umbrella heavy forepoling, made by metallic tubes and grout injections through singles “manchettes”
- Front pre-reinforcement by means of fiberglass nails coupled with grout injections through singles “manchettes”.

The consolidation works and the excavation of the critical zone lasted 7 months. More than 500 m^3 of cement grout were injected (pressure between 3 and 5 bars) into the soil to improve its geotechnical characteristics.

3 PROJECT CHALLENGES

From the Owner’s point of view, the challenges of the project include technical and non-technical issues, such as:

- Being a subway project, *projet m2* is mostly built underground. Project optimization measures brought the portion of the new line which is underground to 90%. This implies much construction work at low depth in an urban environment and the associated level of uncertainty and potential for “bad surprises”. Planning and managing underground excavation and civil works always entails a certain level of risks. (The *projet m2* is no exception, as it suffered by the serious tunnel accident described above).

  Managing this “underground risk” is a classic application of modern risk management thinking. Since “risk zero” is – unfortunately – an unreasonable objective, the stakeholders of underground construction project must be ready to work on measures to limit the probability of negative events and

on measures to reduce their consequences if they occur.
– The civil engineering infrastructure of the project is built by 8 consortia of construction companies working simultaneously on 10 to 12 worksites with 4 sets of engineering firms (fig. 11). The Owner must be in position to manage and coordinate different companies and contracts. Strong presence is required to counter the natural tendency of each individual structure project to "take its own" technical or planning course.
– Because the worksites are located in the urban environment of the city center, special care and emphasis are required on the relationship with the neighbors, public and city services. More than 200 mios € shall be spent in 3 years and a half on the construction side of the project. With such a high rate of worksite production in a relatively small city, it is crucial to control the project impact on the surroundings and the city.

4 THE CLIENT'S TEAM ROLE

The overall project management is provided by the Métro Lausanne–Ouchy SA, a company managed by the public transport authority of Lausanne. Although the engineering services are delegated, as usual, to different consultant companies, the Client has put in place a team specifically assembled to manage the project.

(This section presents "lessons learned" of Owner's representatives on the role of the Owner's team. The authors are fully aware that these comments are nothing new and/or that their relevance to projects conducted in different conditions and context is uncertain).

The client's team is not there to substitute the resident engineers, but to aid and support them in dealing with the strategic issues and specific problems. The tasks of the Client representatives include:

– to support, back up and control the resident engineer in his design and management tasks;
– to coordinate the different lots in order to have consistent application of the generic performance requirements across the project;
– to keep a constant pressure on the resident engineers and contractors, in order to reduce the risk of cost overrun and delays;
– to support the resident engineers for contractors' claims management;
– to centralize information and defend/promote the project toward the public and authorities.

In the end, the primary role of the Client's team is to ensure that the planning and the budget are respected and without compromising on the safety and quality requirements.

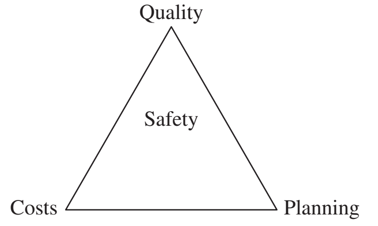

Figure 10. Multi-dimensional approach applied to *projet m2*.

Lots	Engineers	Contractors
2000	**SDIA** *SD Ingénierie, CETP* *Giacomini & Jolliet* *Catella, Hauenstein &* *Ehrensperger*	**CTC 2000** *Batigroup* *Marti et Perrin Frères* *Bertholet & Mathis*
1900		**ATV** **Association Tunnel Vennes** *Infra Tunnel, Bernasconi*
2100	**EMCH & BERGER** **+ LUSCHER**	**JPF Construction SA**
1700	**SDIA** *SD Ingénierie, CETP* *Giacomini & Jolliet* *Catella, Hauenstein &* *Ehrensperger*	**MABAT 1700** *Marti, Batigroup*
1600		**RAM -** *Rusconi, Martin, ADV*
1500	**GEB** *Emch & Berger* *GEOS, DIC* *Architram* *Inst. Géotechnique*	**Consortium Lot 1500 - CHUV** *JPF, Induni, Evéquoz et Gétra*
1400		**Groupement 1400** *PraderLosinger, Losinger Construction,* *Frutiger Vaud, Murer, Déneriaz Sion*
1300	**GEMEL** *GVH Tramelan SA,* *Géotest, Architram* *Fellrath & Bosso SA*	**APB - Association Pont Bessières** *Losinger Construction, Frutiger Vaud*
1200	**GIT-LEB** *Piguet & associés* *CSD-Monod* *Tschumi-Merlini*	**AOC** **Association Ouchy-Croisettes** *Dénériaz, Zschokke Locher* *Grisoni Zaugg, Zschokke* *Walo Bertschinger*
1100		
1030	**SDIA** *SD Ingénierie* *CETP* *Giacomini & Jolliet* *Catella, Hauenstein &* *Ehrensperger*	
1020		**Consortium Lot 1020** *JPF, Frutiger Vaud*
1010		**Consortium RAM** *Rusconi-Martin-ADV*

Figure 11. Overview of *projet m2* engineers and contractors JV.

Since the beginning of construction, the Owner's team has tried to pursue with the Engineers and Contractors a sustained search for solutions in terms of technique, planning and budget. It is clear to all that on a large public construction project, the Owner has to do more then record decisions and events and process payments. He has a central role to play in assisting and guiding the project partners in this optimization effort. He is best positioned to arbitrate between conflicting demands.

Without interfering with safety aspects of the projects for which the specialist engineers and contractors are responsible, he can foster a project culture in which design, problem-solving and decision making explicitly happen in the quality-planning-costs triangle (fig. 10).

One-dimensional problems are the exception; it is usually beneficial to try to explicitly address problems in the three-dimensional. Optimization is about having the best combination of all three. Identifying and evaluating possible trade-offs is difficult, but it is often the key to finding the "best" solution, i.e. to optimization.

Owners' representatives have to be pragmatic and remind themselves and their partners the simple message that: "*The people who finance the whole project does not want the best technical solution at an unreasonable price and unacceptable delay. The people want a reasonable solution at the lowest possible price and in the shortest possible time*".

This is illustrated in fig. 10 with a triangle bringing together the three key elements of project management. Meeting technical requirements (in terms of quality, performance et durability), controlling costs and respecting schedules cannot be dissociated. It is not enough to "have one or the other". Of course, safety considerations must be treated differently because they include non-negotiable elements. But if safety requirements are clearly identified, there is usually a range of solutions to a given problem which respect them and optimization in the quality-planning-costs triangle still applies.

5 SUMMARY AND CONCLUDING REMARKS

Two and half years after beginning of civil works, the construction of the *projet m2* new infrastructure generally progresses according to plan. In spite of a serious tunneling accident described above, the construction project is on time. At this point:

- eight new tunnels have been broken through;
- two 19th century tunnels have been retrofitted;
- more than 1 km of new cut-and-cover trenches have been built;
- two new bridges have been completed.

As any large infrastructure project in an urban environment, especially when underground work is required, the *projet m2* presented interesting and unique technical challenges. Two examples involving the retrofitting of traditional existing structures of traditional masonry construction are described above:

- the construction of a new reinforced concrete bridge through the masonry piles of an existing bridge;
- The underpinning of the lining foundations of a 19th century old tunnel in order to lower its ground slab.

But technical successes are not enough. Unlike scientists who can often focus on technical aspects of their work, our task as engineers has to integrate a broad, and sometimes contradictory, set of requirements and considerations. In particular, the successful completion of large projects – such as *projet m2* – requires the project partners to constantly think and optimize in the quality-planning-costs triangle. In the eyes of an Owner's representative, this approach is as important as meeting the technical challenges. The Owner must therefore foster a project culture and environment where this optimization can happen. Most engineers know that this is part of their responsibility toward the entities they serve and respond well to this challenge.

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Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina & Romancov (eds)
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Metro Santiago – Building an urban tunneling industry from the ground up

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ABSTRACT: Santiago's Metro construction started in the early 70s with the use of extensive cut-&-cover tunnel methods. This type of construction was further used in the 80s, causing major disruptions in the urban surface and strong public opposition. This situation and Metro S.A. key decision to seek for fastest, more efficient and less disruptive construction alternatives, lead to the first mined tunnel experiment under a public Park in 1994. That successful trial tunnel demonstrated the feasibility and benefits of mined tunneling and bestowed Metro S.A. the required confidence to give priority to mined construction in its future projects. Following this decision, and boosted by the continuous and massive array of new subway projects developed in the following decade, a brand-new urban tunneling culture and industry surged in Santiago de Chile. This paper presents this historical evolution, addressing the most relevant landmarks along this way so far.

1 INTRODUCTION

In the early year 2000, the Santiago's subway network was comprised of not more than 40 km and 46 stations with only about half percent of its network being of underground construction, mainly with cut & cover boxes. Nowadays, with the additional 17.5 km of new line extensions planned to be completed in 2009, the subway network will have shown a considerable expansion reaching a total length of 104.5 km and 108 stations, with over 60 km of mined tunnels and more than 70 underground stations.

This lapse of time has seen not only the subway network increase more than twofold but also the establishment, from the ground up, of a whole new urban tunneling industry in Chile. Matter of fact, previously to this period, mined tunneling proficiency in Chile was mainly coming from the construction of rock tunnels for the energy and mining sectors and there was no expertise at all in urban tunneling and soft ground.

Relying mainly on local contractors and designers, fed by a continuous know-how transfer from international tunneling consultants, a remarkably rapid evolution occurred in various aspects of tunnel design and construction, enabling in a short period the development of technical- and very cost-effective solutions for urban mined tunneling in the mostly favorable ground conditions of Santiago de Chile.

Figure 1. Metro system map.

2 SANTIAGO'S SUBWAY HISTORICAL EVOLUTION

2.1 *The early years – 70s and 80s*

Santiago's subway construction began in the early 70s with Lines 1 and 2. Construction was mainly of

Figure 2. Metro construction in the 70–80s – C&C method.

cut- & -cover type with some sections at grade. Cut & cover boxes were built either within sloped or braced excavations. Braced diaphragm walls were also applied at some sections.

This method was further used in the 80s. However, aside from the massive utilities relocation work required, the large construction periods and traffic and surface disruptions, these methods became slowly unacceptable to the community expectations, requiring Metro S.A. to search for less disruptive construction methods (Mercado, C. et al, 2004).

2.2 *The first mined tunnel trial*

In 1994, Metro S.A. ventured to carry out the first mined tunnel experiment, under the public Baquedano Park, during the construction of the first L5 extension. Excavation of the single-tube, double track running tunnel was carried out with full face and the provision of a large central supporting core. Equipment were standard back-hoe excavators.

Design was carried out by local consultant joint venture Ingendesa/ARA. The tunnel presented a horseshoe shape without invert and was excavated in sequential rounds of 0.8 m–1.0 m with a timely differed installation of a double shotcrete lining, 55 cm thick, reinforced with wire-mesh and lattice girders. It is relevant to mention that, at this time, shotcrete technology was poorly known in Chile and only the dry method basics were know from the mining sector experience.

During the construction of this tunnel, monitoring of deformations was carried out only in specific locations and the guarantee of tunnel stability was provided only by the thick shotcrete lining. Ground conditions were favorable, with the typical Santiago's subsoil consisting in quaternary sediments of gravel from the Mapocho and Maipo river basins, the so called "Ripio de Santiago", which presents excellent geomechanical characteristics and are very suitable for tunneling

Figure 3. View excavation and support – First mined tunnel.

(average values at tunnel springline: $\gamma = 22\,\text{kN/m}^3$; $c' = 40$ Kpa; $\phi' = 45°$; $E' = 300$ Mpa; $ko' = 0.3$). The ground water table is variable, but in general is located well below the tunnels at a depth of 70–80 m.

2.3 *Mined tunnels go downtown*

The 2.8 km second line 5 extension to the west was planned to run through a densely built up area in the city center, passing below busy avenues and adjacent to Santiago's old Cathedral and other important historic building. For this endeavor, which would be decisive for the future of mined construction in Santiago, Metro S.A. requested local assigned consultant Cade-Idepe to get support from foreign tunneling experts. For this, Geoconsult of Austria was called in to help with the design of running mined tunnels.

A comprehensive and robust tunnel design was developed by Geoconsult/Cade Idepe with basis on the principles of the NATM (New Austrian Tunneling Method). Soil conditions were the typical Ripio de Santiago, with presence of local ground water infiltration. Tunnel cover was typical with round surface about 10–11 m above the crown. In view of the lack of previous relevant experience, the noviceship of contractors and the sensitivity of the surrounding buildings and structures, running tunnels were intentionally designed as conventional NATM for soft ground conditions.

The tunnel transversal section was ovoidal with an area of about 65 m^2. Tunnel support comprised of a primary shotcrete lining of 200–250 mm, installed immediately after each round, followed by a secondary in situ reinforced watertight concrete lining, with a thickness of 40–45 cm, cast to the invert and arches in a later stage. Excavation was carried out in 0.8–1.4 m rounds with the closed invert no more than 7 rounds behind the top heading. In some sections, micro-piles curtains were installed next to relevant buildings to control deformations.

Stations were constructed as open cuts, but incorporated for the first time the use of ground anchors to support the piles and soil nailing, avoiding the need for the disturbing internal bracing. Even so, very large ground settlements and adjacent building damage was registered within the area of influence of these excavations.

Tunnel construction was carried out from 1997 to 2000 with the incorporation of systematic monitoring of deformations and stresses in tunnel and surrounding ground. Results from this monitoring program showed that ground behavior was very favorable with low convergences and ground surface deformations, mostly in the range of 4–8 mm.

2.4 *Mined tunnels get established – The first mined stations*

In view of the favorable results obtained in the line 5 and the practically inexistent impact on the urban surface and structures, Metro S.A. decided to apply extensive mined tunneling methods on the following subways expansions.

Therefore, for the new Line 2 and 5 extensions, with a total length of more than 6 km, mined methods were applied not only for the running tunnels but also for station tunnels. Construction was carried out from 2001 to 2003. Design and site supervision work was carried out by 3 different JVs of local with international consultants, namely Cade-Idepe with Geoconsult Austria, Ingendesa-ARA with Dr Sauer Co. from USA and Arcadis-Geotecnica with Bureau de projetos from Brazil.

These new extensions marked the return of the use of shotcrete for both primary and secondary tunnel linings. The tunnel lining concept was based on the consideration of a double and monolithic tunnel lining, where both linings would work together to provide the long term tunnel support and service conditions. This was possible basically due to the favorable ground conditions of the Ripio de Santiago, the inexistence of significant ground water and application of state-of-the-art shotcrete technology, including low dosages of alkali-free accelerators, additives, meticulous shotcrete installation, curing and use of steel fibers to increase quality of the secondary lining.

For the running tunnel sections, horse-shoe tunnels (without invert) were extensively applied and the thickness of the primary lining was reduced to only 15 cm. Tunnels with structural ring closure were applied only at singular sections, such as intersections or heavily loaded sectors.

Stations were typically constituted by one or two lateral shafts, located in a place as to not interfere in public roadways and require extensive utilities relocation, and associated access tunnels, placed mainly in a direction transversal to the main line alignment so that it crosses in a perpendicular way with the main station tunnel.

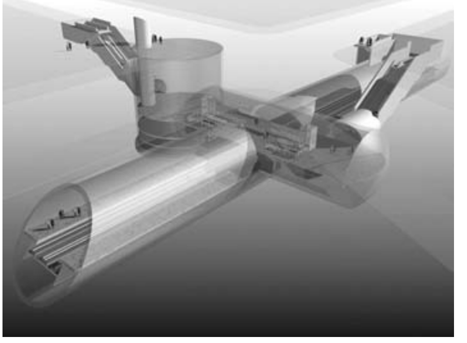

Figure 4. View of a mined station tunnels.

Figure 5. Excavation of access tunnel under pipe roof.

Station tunnel cross sections varied from 100 m^2 at intermediate stations to up to 180 m^2 at terminal or multi-modal stations. With rail levels at a depth of 17–18 m, the crown of cross sections up to 17 m wide were only 6–7 m below the ground surface.

Access tunnels were constructed under the protection of a pipe roof umbrella, as to control ground deformations and allow full-face exaction of the top heading section, which was completely excavated and supported, including a temporary invert, before carrying out the excavation of the bench and invert headings.

After the access tunnel excavation was finished, the break-out for the excavation of the main station tunnel was done. The station tunnel was constructed subdividing the section in 3 or 2 separated headings with the provision of temporary lateral walls, demolished as the excavation stages advanced. No pipe roof or forepoling was installed for these sections. Significant lessons regarding design and construction aspects were drawn from the construction of these two extensions. The most relevant are listed below:

– ground conditions response to the excavation of large dimension tunnels were favorable and safe (max. settlements were in the order of 2 cm);

Figure 6. View of access station tunnel end wall.

Figure 7. View of a mined station tunnel excavation.

- stresses on the linings were below those foreseen in the design, allowing the further optimization of tunnel support;
- The control of shotcrete technology and installation was proven critic to ensure the required lining quality;
- Efforts should be made to reduce the amount of reinforcement in shotcrete linings, as there was a clear limit to get rebars fully embedded and obtain good quality linings, avoiding the need to carry out extensive repair works by post-injection;
- It was not possible to obtain watertight linings with the use of shotcrete (the installation of a sprayable waterproofing membrane between the two linings was proposed in the design but not applied during construction);
- The provision of a qualified and expert engineering team (Site Supervision) on site is of most relevance for tunneling works, providing vital control and solutions to guarantee safety, construction schedule and the specified quality.

2.5 *Tunneling under tunnels and old bridges*

Another line 2 extension's landmark was the construction of a tunnel section below the bottom slab of

Figure 8. Interior view of a finished mined station in shotcrete.

the six-lane cut & cover highway tunnel "Costanera Norte" and of a large subway mined tunnel section directly under the abutment foundation of an old bridge for vehicular traffic across the Mapocho River, in the centre of the city (Gomes, A., Boefer, M. & Cruz, J.H. 2004).

Whereas the passage below the cut & cover tunnel constituted no major challenge, the crossing below the bridge abutment required special construction methods and additional auxiliary measures to allow tunnel construction and prevent damages to the bridge. These included the underpinning of the bridge abutment, the installation of an active jacking system, the use of a subdivided excavation face and the implementation of a comprehensive and continuous monitoring system of loads and deformations, designed specifically for this project.

2.6 *New grounds and wet shotcrete*

With the new line 4, constructed from 2003 to 2005, Metro S.A, started one of its more impressive investments in the subway network. The new line encompassed app. 32–33 km, with 12 km of mined tunnels, 14 km at grade and 7 km of elevated rail.

Since its construction overlapped with the construction of the Line 2 and 5 extensions, there was a market peak, which generated a certain overstrain in the local construction industry. Nevertheless, this also helped to create the right scenario for major improvements in technology and a doorway for new local and foreign contractors and companies.

Metro S.A. used the line 4 to foster the use of more advanced techniques for tunnel construction. Therefore, the JV Geoconsult-Cade Idepe was hired to carry out a comprehensive feasibility study on the use of mechanized tunneling (TBM). By using the typical NATM design as a baseline, a comparative study was carried out for different TBM alternatives. This study

Figure 9. Tunneling below the Fray Andresito bridge.

concluded that the NATM approach was the most appropriated approach for the line 4 underground works, as it was less expensive, generated more benefits to the local industry and posed fewer risks for the project schedule.

Another important decision taken by Metro S.A. at the new line 4 was the specification of state-of-the art wet shotcrete for tunnel support, allowing the further development of this technology in Santiago.

The construction of the line 4 also marked the first tunnel excavation in soft ground and the harshest handling of ground water at the Station Puente Alto (Gomes, A. & Boefer, M. 2005). Actually, a large part of the alignment crossed sediments deposited by erosive streams from the Andean Cordillera, consisting mainly of clay and silt with low to moderate plasticity and sand lenses with different thickness. In Puente Alto, even tough the ground was the typical and favorable Ripio de Santiago, the presence of water inflow (up to 15 l/s) from water bearing layers, associated with the contractor inexperience caused important construction problems and program delay. Problems of ground water were also found in other sections of the line 4, without causing many difficulties.

Design concepts were kept basically similar to those used for previous lines. However, some new construction methods were applied, such as the design of the Plaza Egaña station with the use of a composite geometry with double cell and a central pillar, as to reduce excavation area, avoid sinking of rail level and limit deformations to acceptable values. Besides, some sectors presented local instability problems required the establishment of a comprehensive and monitored excavation procedure to control local ground collapse and reduce the risk of global cave-ins. This was managed by the provision of geologists at the face, under the supervision of the site Engineer and in close work with the contractor. Another change was the consideration of an in situ inner concrete lining for the Station Puente Alto due to the presence of ground water.

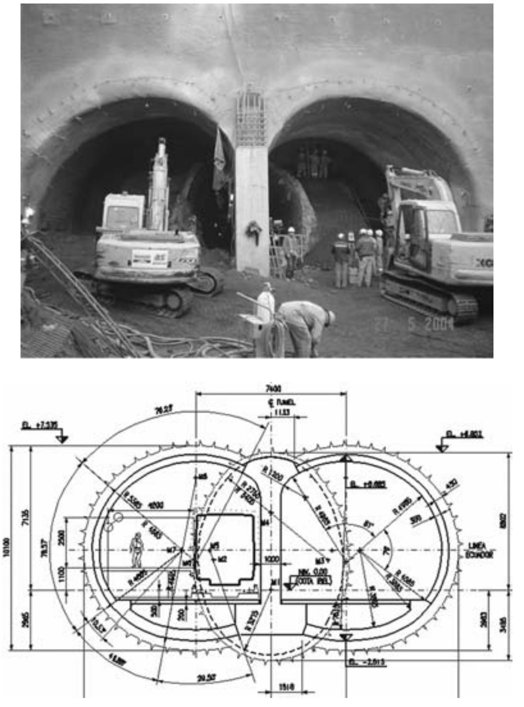

Figure 10. View of station tunnel with central pillar solution.

2.7 *More soft grounds and further expansion*

In the 13 km new line 2 extension to the North, constructed in 2004–2005, aside from typical Ripio de Santiago conditions, tunnels were also excavated in soft alluvial ground (clay, silt and sand layers, intercalated with sand lenses and gravels). Similarly to the experienced in the line 4, local instability problems required the establishment of a comprehensive and monitored excavation procedure.

For this extension, a significant optimization of the lining design was achieved, reducing both thickness and amount of reinforcement in station tunnel crossings.

2.8 *The latest expansions*

With construction start planned for the beginning of 2007, the new extensions of Line 1 to the East and Line 5 to Maipu (West): 17.5 km, will also shown some novelties and new challenges to designers and contractors. At the line 1, ground conditions are the typical Ripio de Santiago. Aside for some tunnel crossing and other singular situations, no significant novelty is expected. However, at the Line 5 to Maipu, tunnels will cross not only the typical fluvial deposits of gravel but also other ground conditions where there is no previous tunneling experience.

The most challenging will be the sectors with raveling gravel, which may require special pretreatment of the excavated surfaces, prior to installation of the shotcrete lining. There will also be sectors running through fine silt-sands deposits of volcanic origin with low cementation, the sol-called "Pumicita", and mixed soft ground conditions with the presence of clay, silt, sand and gravel layers. Additionally, a tunnel sector may experience relevant ground water inflow, requiring proper water control. At the time of preparing this paper, the design was still being carried out. However, even tough not all particularities are defined yet, it can be anticipated that the design of these new structures will incorporate many lessons learned in the past, providing even more optimized solutions, such as the flexibilization of invert installation to account for the different ground conditions.

3 PROCUREMENT AND FINANCING

3.1 *Procurement*

The typical approach to design and construction is based on the development of basic and detail designs by consultants hired by Metro S.A. and bid and build contracts. Bidders are pre-qualified contractors and contracts are awarded to the lowest technically complying bid. Contracts are administrated by Metro S.A. managers, together with a dedicated site Inspection, also provided by local consultants. Additionally, the site engineering supervision, which oversees the overall tunnel construction, is granted to consultants together and as an extension of the detail design works carry out for each contract.

3.2 *Financing*

Since 1990 Metro S.A. is an independent government-owned institutions that own, operate and maintain the subway system. In general, costs of civil works are borne by the National budget through funds granted by the Ministry of Finance. M&E and rolling stock costs are generally financed directly by Metro S.A. itself by means of loans lent by the international banking system. Regarding operation costs, those are totally covered by Metro S.A. own revenues.

4 COSTRUCTION COSTS, PRODUCTIVITY

During this decade of development, construction costs have decrease and productivity increased significantly for the Metro de Santiago underground works. During the first years of tunneling, average advance rate of running tunnel excavation was inferior to or maximum 1.0 m daily. Nowadays, average advance rate reaches up to 2.5 m daily.

Costs have also decreased substantially. Initial ruining tunnel costs per lineal meter were in the range of U$15.000–20.000. Currently, these costs lay typically in the range of U$7.000–8.000. Another important cost reduction aspect was the continuous optimization of the monitoring program, as more knowledge of the ground behavior became available. This effort allowed a reduction from the initial cost of U$1million/km to the current figures, which are lower than U$250.000/km.

5 CONCLUSIONS

On the basis of a successive array of investments in new subway projects and a strong compromise to pursue low disturbance, cost-effective and pragmatic technical solutions, Metro S.A. has managed to develop in a short period of time a remarkable, safe and efficient subway network, which shall still continue to expand in the future.

In this quest, Metro S.A. has also helped to forge a new tunneling industry with qualified and competent tunneling contractors, consultants, suppliers and workers, able to respond to Metro S.A. needs to further develop its subway network, which constitutes the backbone of the integrated Urban Transportation Plan of Santiago (UTPS).

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The Slivenec Tunnel on the Prague City Ring Road

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ABSTRACT: A motorway-type ring road is being developed to reduce traffic in Prague. The southern section is currently under construction. The route in this section traverses the Vltava and Berounka valley, to which it descends from a quite steeply rising hilly country along both river banks. In addition, there is a protected landscape area there. For this reason, the authors of the design have proposed underground sections for this locality (construction lots no. 513 Lahovice-Vestec and no. 514 Lahovice-Slivenec). These sections consist of uni-directional tunnels, with two lanes in descending sections and three lanes in ascending ones. In addition, given the steep 4% gradient, the ascending sections are provided with a slow lane for trucks. The tunnel safety equipment is on the top worldwide level. In order to verify the geological conditions thoroughly, an exploration gallery has been designed to pass through the entire three-lane tunnel section. The position and shape of this gallery have been selected in such a way that the gallery will substitute the initial tunnel excavation sequence, the top heading. The gallery bottom will be identical with the floor of the tunnel top heading. The exploration gallery situated in this way will not only serve its primary exploration purpose. The site facility built for the works will even be utilised for the future tunnel excavation. The concept and the technical design of the tunnel sections meet the latest EU safety regulations. The Slivenec tunnel is 1,610 m long in total.

1 THE ABSENCE OF A RING ROAD AROUND PRAGUE AND ITS CONSEQUENCES

The number of vehicles significantly grew in recent years in the Czech Republic. The high traffic volume, unusual in the past, is obvious everywhere. If you look at a road map of the Czech Republic, you will see the apparent centralism in the layout of main roads and motorways; the majority is heading to Prague.

Today's Prague, however, has changed for many drivers from a target city to a transit city. This fact can be related to the political change of the formerly closed socialist state to a state with a democratic system, which is open for the whole Europe. The current large proportion of international transit is one of the effects. International transit routes cross the Czech Republic from Germany in the southwest or northwest to Poland and Slovakia in the east, from Austria in the southeast to Germany in the northwest, etc. The vehicles only passing through Prague put an unnecessary burden on Prague streets and municipal roads. In rush hours, this situation is often unbearable. A ring road around Prague is being badly missed.

The main task of the ring road will be to reduce the traffic volume within the network of Prague streets and local roads, and interconnect all motorways and fast highways before entering Prague, thus to allow the transit transportation to switch fluently between all long-distance radial routes leading from Prague. Considering the location of our republic in the centre of Europe and with respect to the planned cancellation of borders (incorporation of the Czech Republic into the union of Schengen countries), and owing to the actual process of completing the basic network of our motorways, a considerable volume of transit through our territory can be expected in the years to come. The increase in the proportion of truck transit which was recorded after the accession of the Czech Republic to the EU in May 2004 has confirmed the anticipation that all of this traffic will prefer the PRAGUE CITY RING ROAD to the problematic and slow passage through the centre of Prague.

This fact has become clear to members of parliament, mayors, directors of big companies, investors, designers, contractors and all other people who would like to promote the development of the ring road. They have founded the Prague City Ring Road Development Association, which has been creating space for dialogue and, lately, has been helping to accelerate the work on the design and construction of the ring road.

2 THE PRAGUE CITY RING ROAD HISTORY

The first ideas of the Prague ring road project are dated the 1920s. This period even saw first efforts for development of a general plan of the traffic solution for Prague.

Other proposals for the development of a ring road around Prague emerged before World War II. They were based on a contention that "the city of this magnitude cannot cope without a project like this". The ring road, as a basic principle of diverting transit transportation to bypass the centre of the city and to interconnect city centres found in the outskirts was the subject of numerous studies carried out in Prague.

However, the construction of the ring road was approved as late as 1963, and the implementation of the PRAGUE CITY RING ROAD started in the 1980s. The decision that the ring road will be only of the fast highway type instead of the motorway type was made in 1987. This decision also meant the end of discussions about the number this motorway was to be assigned (the D1 motorway had already existed then).

3 THE DESIGN OF THE HORIZONTAL ALIGNMENT AND CONSTRUCTION OF THE SOUTHWESTERN PART OF THE PRAGUE CITY RING ROAD

The process of designing the alignment of each of the segments of the Prague ring road has always risen lots of questions and problems, which were solved and analysed in studies and designs carried out in the particular period. The pressing character of the task to solve the problem became fully obvious only in the 1990s, in the context of the social changes after 1989 and the enormous increase in the vehicular traffic volume associated with this process. Today, there are two parts of the PRAGUE CITY RING ROAD in operation, namely the Slivenec – Ruzyně section (construction lots No. 515, 516 and 517) on the west edge of the capital, and the Satalice – Běchovice section (construction lot No. 510) on the eastern edge.

The most pressing traffic situation is currently on the Barrandov Bridge, where traffic jams even last over 12 hours in a day. The entire length of the Southern Connection Road, which is part of the inner City Circle Road, is filled with standing vehicles. Each driver can experience there the passage through a big city lacking a road loop. The traffic volume on this road, which is today one of the roads most loaded by traffic, should be reduced by the south-western part of the ring road consisting of the construction lots No. 513, 514 and 516, which together form a road link between the D1 motorway for Brno and the D5 motorway for Plzeň. The alignment of the southern part of the Prague City Ring Road crosses the valley formed jointly by the Vltava and Berounka Rivers. It falls to the valley from a relatively steeply rising hilly area lining the river banks. For this reason, and also because of the fact that it passes through protected areas, the alignment of the sections of the construction lot No. 513, Lahovice – Vestec, and construction lot No. 514, Lahovice – Slivenec, is designed to pass through tunnels. The right-hand tunnel tubes are offset from the centre line of the route by means of radii slightly differing from the radii of the route so that an optimum distance between the tubes is achieved.

4 VERTICAL ALIGNMENT AND DIFFERENCES IN THE CONFIGURATION OF TRAFFIC LANES

The revised norm currently valid for road tunnels recommends gradients up to 3.0%; tunnels with steeper gradients need an assessment to be performed whether additional safety measures are necessary. The 4% gradient at the lengths over 1.5 km means, in our specific case, that an additional a traffic lane for slow moving vehicles and trucks is necessary.

Traffic engineering measurements showed that freight traffic makes up a 40% proportion of the total traffic volume, while a 15% proportion had been assumed before the measurements. From this viewpoint, the addition of one traffic lane at an uphill section is obviously necessary even for a smaller gradient. For this reason, three-lane tunnels are designed for the tubes carrying the traffic ascending from the confluence of the rivers up the plateaus. The solution of the question whether the capacity of the 4% uphill gradient three-lane roads is sufficient will remain to be answered in the future. The descending tunnel tubes have two traffic lanes.

The width of the right traffic lanes of 3.75 m, which is increased compared with the standard width of 3.50 m, was designed with respect to the significant proportion of truck traffic. The widths of the roadway were binding for the designer.

5 ARCHITECTURAL DESIGN OF THE PORTALS

Tunnel portals are nearly always difficult items of any tunnel construction. Even the choice of their location depends on many factors, mainly on the terrain configuration, the horizontal and vertical alignment of the route, geological conditions in the particular location, proper incorporation of the structure into the landscape and other factors. All of the above circumstances also apply to the portals of the tunnels built on the Prague ring road. The Lochkov portal of the construction lot No. 514 belongs among the most difficult. It is located on a steep slope above Radotín, where the nappe (gravelly soils) is over 10 m thick. In this case, the portal of the mined tunnel is properly shifted as deep to the rock massif as possible, to a point where at least the bottom part of the tunnel will be excavated in rock conditions. When you walk in this area you can see nothing exceptional. The terrain above the future tunnel, a meadow with naturally seeded trees and brushes, looks subdued. Unfortunately, there is a biological protected zone located just in the place most suitable for the portal. It means that the initial section of the tunnel must be shifted to a place where, simply put, the conditions for tunnelling are the most difficult. The design of the construction trenches for the portals of the mined tunnels comprises mighty vertical piles keyed in the bedrock, with the steep slopes stabilized by shotcrete, anchors and needles. The trenches will be used for the construction of both the cut-and-cover tunnel sections and the architectural concrete portals. The C25/30 reinforced concrete lining of the cut-and-cover tunnels is 600 mm thick at the vault. The waterproofing system applied to the cut-and-cover tunnels consists of a membrane installed on the outer surface, which is protected on both sides by non-woven geotextile. The cut-and-cover tunnels are founded on strip footings; the invert will be built at the portal.

All portals are designed as cast-in-situ reinforced concrete structures. The elliptic front ends of the portals of the construction lot No. 514 are provided with "ribbed canopies", which improve the architectural impression of the portals and help to incorporate them into the landscape.

This architectural design is used for tunnels designed in our country for the first time and, as you can judge by yourselves, it certainly will make the passage along this section of the route brighter. The finish of the terrain around the portals will be created by the backfill of the adjacent cut-and-cover tunnels, and by a part of portals inclined approximately at 1:1.75. The side slopes of the portals are terminated by retaining walls and service buildings, which are incorporated into the walls.

The chimney of the ventilation structure is also designed as an architectural feature.

6 MINED TUNNELS, CROSS SECTIONS THROUGH TUNNEL TUBES AND CROSS PASSAGES

Mined structures (tunnels, cross passages etc.) form a substantial proportion of the construction lots No. 513 and 514 of the road ring. In terms of the structure, all of the tunnel structures are similar, consisting of reinforced C 20/25 sprayed concrete primary lining, cast-in-situ C 25/30 concrete final lining and intermediate waterproofing layer. The vaults of both liners are circular. All of the tunnels will be driven using the New Austrian Tunnelling Method with a horizontal excavation sequence. The excavated cross-section areas of the double-lane tunnel and three-lane tunnel will be 95 m^2 and 130 m^2 respectively. The internal profiles of the tunnel tubes are vaulted, uniform along the entire lengths, irrespective of the construction technique.

Walkways 1.0 m wide are provided on both sides of the roadway. The height clearance is 4.80 m; higher vehicles could pass through the tunnel exceptionally if they ride under the middle of the vault.

7 SAFETY IN TUNNELS – TUNNEL EQUIPMENT

The scope of safety equipment of road tunnels is wider than that of railway tunnels because of the fact that much more subjects (vehicles) move in a road tunnel, the freedom of their movement is much higher, thus the chance that a dangerous situation will occur during the operation is much greater.

The safety equipment meets the ČSN 73 7507 norm and is designed in compliance with the Directive of the European Parliament and of the Council on minimum

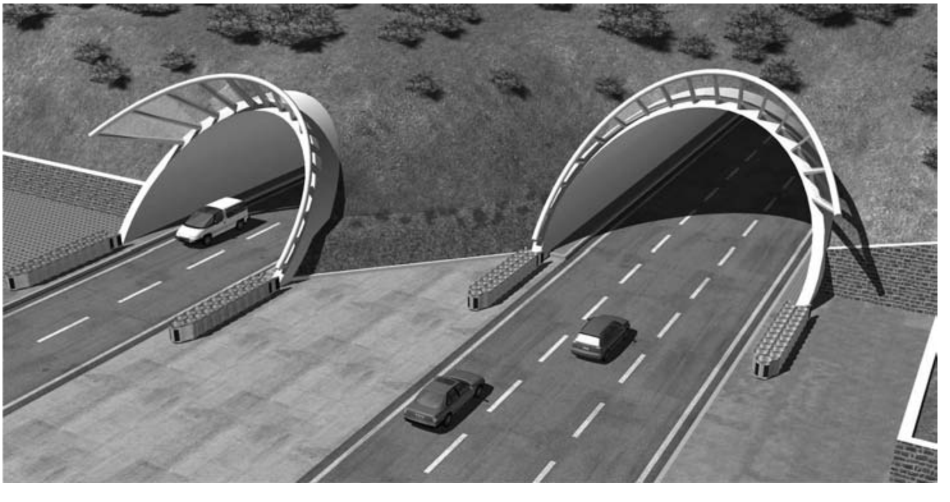

Figure 1. Architectural image of the portals.

safety requirements for tunnels in the Trans – European road network.

The recent conflagrations in European tunnels have shown us that the possibility of escaping from the tunnel is the most important factor of safety in a tunnel. This possibility is provided by construction of the above-mentioned cross passages, which are always equipped with fire resisting doors and separate ventilation systems. In addition, there is an emergency lay-by in each double-lane tunnel to be used for stopping and parking immobile vehicles; the lay-bys also facilitate contingent exiting of trucks from the cross passages passable for vehicles to the double-lane tunnels.

SOS boxes are installed every 150 m. They contain telephones, fire extinguishers and other equipment. A fire main with big hydrants, which is intended for the use only by firemen, is installed in each tunnel.

Of course, tunnel illumination with threshold and transition zones at portals, where the gradual reduction/increase in the lighting intensity provides gradual darkening/lightening and driver eyes can accommodate at the tunnel entry/exit, are a design standard. Stand-by lighting to be used in the case of an accident or fire is also installed in each tunnel.

The traffic control will be possible by variable traffic signs installed both inside the tunnels and on the open road before the tunnels. The tunnel control system is connected to the Traffic Information System and the data is transferred to control centres in Rudná and Prague, where the traffic data collected from the route is assessed. The processes in the tunnel will be observed by video cameras.

The tunnels are ventilated by jet fans installed under the tunnel crown at intervals about 200 m. In the basic regime, they are operated in the direction of traffic movement. The position and output of the fans is designed with respect to a fire emergency. The ventilation will be started during a fire after roughly 6 minutes to 8 minutes, when the cooling of smoke under the tunnel arch causes destratification of the smoke, which starts to threaten the passengers. The hot smoke keeps under the arch till that time and the escape of passengers is possible. The fans in the neighbouring smoke-free tunnel tube will be started in the same direction as those in the tunnel with the fire so that the smoke and other pollutants are not sucked to the clean tunnel via the portals. A ventilation plant for an exhaust ventilation system is designed, which will provide the required dispersion of pollutants in the right-hand (ascending) tunnel tube and partially at the beginning of the left tube.

The power consumption of all of the above-mentioned facilities is very high.

A transformer station and distribution substations are installed in single-storey buildings located in front of the portals; points where the electrical installations in the tunnels can be disconnected are available every 400 m. The power supply is backed up in case of a power failure.

Even the roadway drainage is designed with safety taken into consideration. Inverted siphons preventing a fire caused by an accident from spreading interrupt the drainage ducts installed along the curbs every 50 metres.

8 GEOLOGICAL CONDITIONS

The tunnels of the construction lot No. 514 will be driven through Ordovician and Silurian rocks, which are tectonically faulted and have pervasive faults and folds. Heavy fracturing of the rock occurs along the

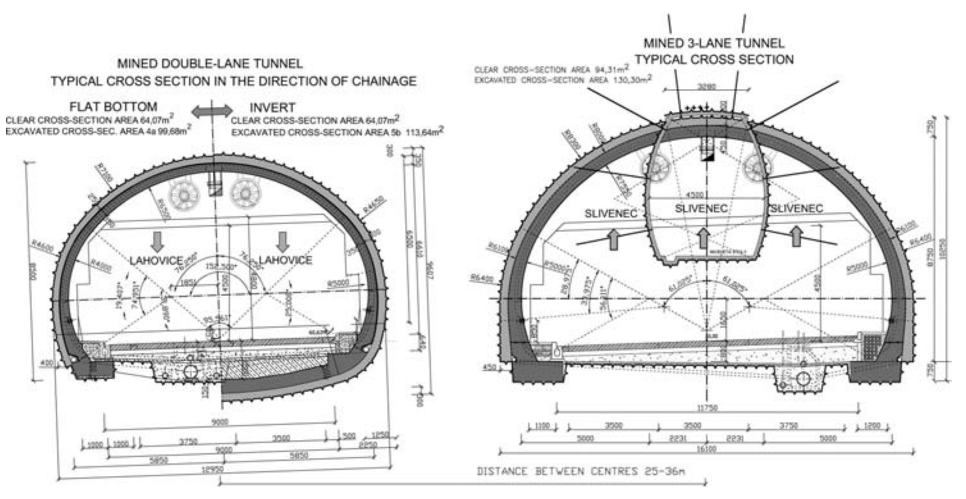

Figure 2. Cross sections.

faults. The shales, which are crushed by remoulding, are slickensided. Fissures are frequently open or partially filled with crushed rock and water. The numerous faults vary in their trends; the NE – SW trend dipping 45° to 60° to northwest can be considered prevailing. However, even transverse down-faults sub-parallel with the route are frequent. The filling of the down-faults consists of breccias of a character of shale fragments or clay. The information about quantity and quality of the faults was refined by an exploration gallery, which was driven along the central line of the top heading of the three-lane tunnel tube.

The hydrogeological conditions are relatively favourable for the excavation despite the fact that nearly the entire length of the mined tunnels passes under the water table lowered by the drainage effect of the exploration gallery. The aquifer existing in the area of the mined tunnels is bound to a fissure-type collector in the weathered zones of the bedrock. The water table copies to a certain extent the ground surface. The yield of the inflows at the excavation face may reach tenths of litres per second, in the case of quartzite or diabase it may grow to roughly 1 l/s, and in the case of water bearing faults up to 3 l/s.

9 THE EXPLORATION GALLERY – A PRE-CONSTRUCTION ELEMENT IN THE AREA

Geological conditions anticipated according to a geological survey can be verified during the planning phase of larger projects by excavation of an exploration gallery.

The exploration gallery, however, is always a matter of discussion in terms of the tunnel excavation; pros and cons of exploration drifts are relatively well known.

Today we would like to draw attention to a little bit different aspect. In addition to the exploration of the geology proper, the exploration gallery will test, on a smaller scale, even other relationships, and will “teach” the inhabitants and the entire region how to coexist with the future construction activities. In practice, it means that the condition survey of the affected buildings must be carried out and the blasting permit must be obtained; truck traffic to the construction site will start, thus the haul routes will be tested. Some utility networks will have to be relocated, the land for the site facility will have to be obtained, selected areas paved, power, water and sewerage services built etc. The inhabitants will feel all of these activities similarly as they will feel them during the future tunnel construction, only to a smaller extent. This process will help to forecast their future attitudes and requirements.

The results obtained by the exploration will be available to the designer and the contractor, who will use them in the process of optimisation of the construction procedures and selection of adequate tunnelling equipment.

10 TECHNICALLY SUITABLE SETTING OF THE EXPLORATION GALLERY

Till now, small cross-section exploration galleries have usually been designed with the aim of minimising the cost. Geological conditions have lately ceased to be the main factor in the design of the alignment passing through a more densely populated area. The final alignment is the result of strenuous searching for a “practicable” corridor.

The extent of tunnel projects, their exactingness and cost demands, the requirement for quick and economic excavation using up-to-date high-performance equipment, require broad knowledge of geology as well as knowledge of the rock response to excavation by heavy equipment. For this reason, “hand excavation” of minimum cross-section galleries with imperfect temporary support is today unsuitable.

The utilisation of heavy equipment requires the exploration gallery cross section to be enlarged; the enlargement requires the setting of the gallery to be selected which is optimal with respect to the subsequent tunnelling work so that the in advance excavated exploration gallery interferes with the excavation of the future tunnel as little as possible, and, on the other hand, contributes to facilitation and acceleration of the work.

The issue of the size and setting of the exploration gallery has been step by step solved for tunnels on the Prague City Ring Road.

The first gallery was designed as a traditional small horseshoe shaped profile gallery with the cross section of 13.8 m^2, set in the centre of the top heading of a three-lane tunnel. The aim of this design probably was to prevent the rock disturbance effects from reaching the surface of the future tunnel excavation. Such the setting will influence the further progress of the excavation of the tunnel tubes in a minimum extent, although, it can contribute to the facilitation of the future tunnel excavation also in a minimum extent if we consider the fact that the floor of the gallery is at least 1 m above the expected bottom of the top heading.

Metroprojekt Praha a.s. negotiated with the client a change in the cross section and setting of the exploration gallery when the work on the final design for the tunnels of the construction lots 513 and 514 was commencing. They incorporated a new setting and an enlarged cross section of 25.6 m^2 into the design; the floor of the gallery was designed at the level of the future tunnel. The height of the gallery is an optimum for utilization of high-performance equipment and for its radius of handling at the excavation face. The

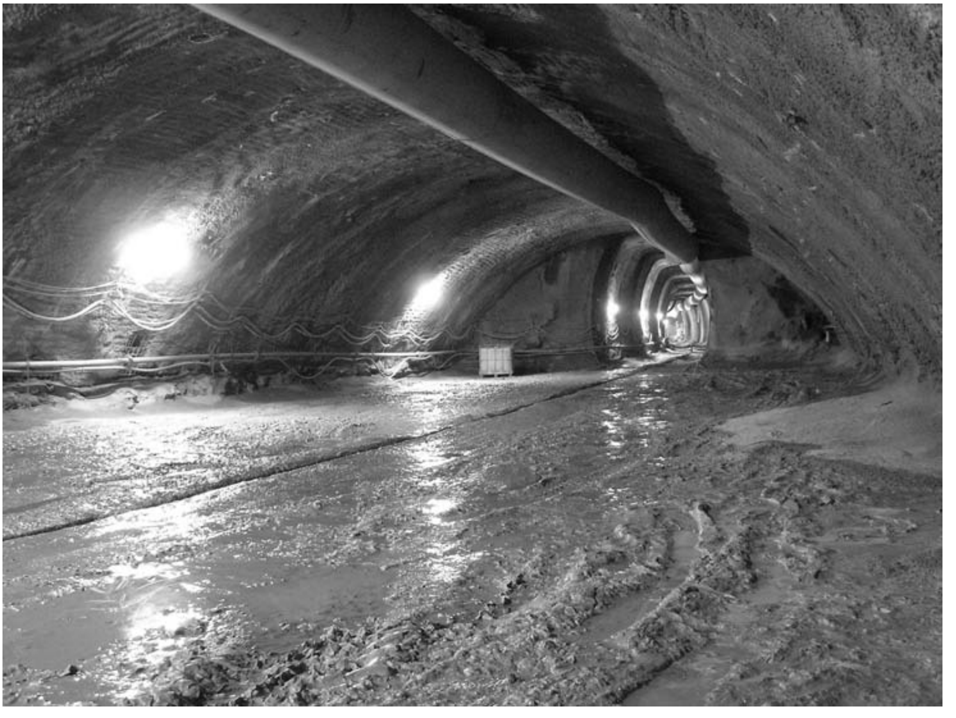

Figure 3. A passing bay in the exploration gallery.

passing bays even represent complete top headings of the tunnel.

The setting and shape of the exploration gallery were selected so that the crown of the primary lining of the gallery extended into the primary lining of the future tunnel and became its part; later, because of the lapse of time between the gallery construction and the tunnel excavation, the gallery arch lining was shifted above the tunnel lining. Owing to this setting, the exploration gallery will create the first partial excavation (sequence) of the top heading of the three-lane tunnel. The primary lining of the gallery will act as a mighty anchor during the course of the top heading excavation. At the same time, it will significantly reduce the excavation face, which will become stable even without any additional measure. This gallery will not deteriorate the conditions for the excavation of the three-lane tunnel proper; just opposite, it will significantly contribute to the tunnel excavation safety.

Apart from its exploratory purpose, the completed exploration gallery, in its new setting, will become an important ventilation adit, escape route and access adit for rescue services in case of an emergency, even an optional supply route. It will allow gravitational drainage of the excavation during the downhill excavation of the large tunnels. The ideal opportunity of installing anchors in the crown of the vault in advance of the tunnel excavation and implementing in advance extraordinary or required measures in locations of weakness zones or other extraordinary conditions identified by the exploration gallery must not remain unmentioned. As a last resort in case of unfavourable rock conditions, a segment of the primary lining of the tunnel can be cast in the crown of the gallery (at a width equal to the width of the gallery) reinforced sufficiently to be capable of acting as a stiff longitudinal beam significantly reducing deformations in the rock mass and surface settlement during the work on the enlargement of the top heading and installation of the excavation support.

11 CONCLUSION

Today, the excavation of the exploration galleries for the 514 and 513 tunnels and the refining the information on geological conditions have been completed. The information and experience gained from the excavation of the exploration galleries have been incorporated into the detailed designs for both tunnels. The excavation of the tunnel 514 has commenced.

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Portals – The architecture visible in cities and landscape

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ABSTRACT: Tunnels are civil engineering works where emphasis is mostly laid on their technical and economic design. The public usually view them as very important, even though unsightly rather than nice; as a structure that only serves its purpose and is no aesthetic complement or enhancement of the landscape in which it is located. In the past, tunnel portals were integrated with the ambience without affecting the ambience significantly, in a somewhat uniform style and often intentionally overshadowed by other structures. With the new upswing of the tunnelling industry in the Czech Republic over the last ten years, even tunnel designers dared involve capable architects, whose task was to complete this technical work so that it not only serves its purpose but its appearance also completes the architecture of the territory, and thus underlines the clear meaning of this large investment. The new approach has even made some of the current portals landmarks of the respective territories. The paper illustrates and describes the architectural concept of the tunnel portals being currently under construction or in the preparations phase, and brings back to mind the most important historic portals in Prague.

1 INTRODUCTION

The focus of this paper is on picturing tunnel portals. Photographs of structures built in the past or visualisation of projects being prepared are provided with brief descriptions.

Figure 1. Rudolph's Gallery, the year 1593.

2 SMALL TUNNELS

2.1 *Rudolph's gallery*

It is an underground gallery leading between the embankment of the river Vltava and Kralovska Obora (Royal Deer Park), which is the oldest tunnel in Prague. This historic monument was built in the years 1581–1593 to supply water to a pond in the deer park. The about 1000 m long gallery was excavated simultaneously from both ends and additional five shafts. The dimensions of the cross section are approximately 3.5 m $\times$ 2.2 m. The shafts and the gallery were supported only with round timber, which was replaced by masonry during the 18th century. This remarkable piece of technical work has been preserved up to the present time.

2.2 *The tunnel under the Prasny Bridge embankment*

A possibility of interconnecting both parts of Jeleni Prikop (Deer Moat) divided by a road embankment was under consideration from 1990. The tunnel has opened the access to another part of Prague Castle. The shape of the new tunnel and portals as well as the whole construction were sensitively designed by

Figure 2. The tunnel under the Prasny Bridge, 1990.

an architect. The route of the passage is identical with a former Theresian gallery allowing Brusnice stream to pass through the embankment.

The new egg-shaped tunnel is 84 m long. Three basic materials were used for the construction: concrete, hard-burnt brick cladding and, for pavements and the stream channel, natural stone.

3 ROAD TUNNELS

3.1 *The tunnel under Vysehrad*

The 34 m long × 9 m wide tunnel was opened to traffic on 11th December 1904.

The tunnel was considered necessary for vehicular and pedestrian traffic, which was a condition for further development of Prague districts of Podoli, Branik and Hodkovicky (villages on the outskirts then). The ambience around Vysehrad, which is a significant, nearly mythical historic monument, has always a romantic air. This was probably why the authors used crenelation and turrets for the portal decoration, which enhances this romantic character.

3.2 *The Letna tunnel*

The first real road tunnel was the Letna tunnel built in 1949–1953. It is 426 m long × 10.3 m wide × 6.5 m high. The gradient of the S-shaped alignment is uniform at 5.4%.

The tunnel links the area of Letna with the centre of Prague. It was excavated using the Modified Austrian Tunnelling Method.

3.3 *The Strahov tunnel*

The Strahov tunnel route connects the areas of Brevnov and Dejvice with the area of Smichov. The history of this connection can be followed in proposals developed

Figure 3. The tunnel under Vysehrad decorated with crenelation; 1904.

Figure 4. The Letna tunnel, 1953.

Figure 5. The Strahov tunnel, 1990.

as long ago as 1847 and 1937. The Strahov tunnel was inaugurated in 1997 as a part of the inner circle road.

The tunnel was driven using the side-drift method, which was probably a little bit archaic at that time.

3.4 *The Mrazovka tunnel*

The Mrazovka tunnel on the City Circle Road in Prague was built recently. The portals have been

Figure 6. The Mrazovka tunnel, 2004.

Figure 7. South-East Portal of The Mrazovka tunnel, 2004.

properly incorporated into the surroundings, from both architectural and town planning viewpoints.

The construction of the Mrazovka tunnel started in the autumn of 1998. It was opened to traffic in August 2004. The tunnel is 1300 m long; three traffic lanes are provided in the major part of its length. The excavated cross-section areas in the three-lane section and in bifurcation chambers were 165 m^2 and up to 340 m^2 respectively. The excavation of the tunnel passing under the relatively densely developed area was carried out by the New Austrian Tunnelling Method.

The northern portal is nearly directly connected to the southern portal of the Strahov tunnel. A cascade of greenery troughs suspended above the portal ends in a park on the top of Mrazovka Hill. The southeastern portal is provided with a staircase leading to a lookout located on the hillside.

3.5 *The Blanka tunnel*

The further development of the City Circle Road in Prague will comprise the Czech Republic's longest road tunnel at a total length of 5.5 km. The construction is planned to begin in 2007.

Figure 8. Troja axonometry – visualisation.

Figure 9. Portals of the Prague City Ring Road – visualisation.

The Blanka tunnel starts at the Malovanka grade-separated intersection, at the northern portal of the Strahov tunnel. In strict sense, the only real portal is the Troja portal.

3.6 *A portal and ventilation shaft of a tunnel on the Prague City Ring Road*

The construction lot 514 of the Prague City (outer) Ring Road project comprises tunnel tubes with "ribbed canopies" skirting the elliptic crowns of the portal ends. This architectural design will certainly relieve the dullness of the passage along this section of the route.

3.7 *The Pisarky tunnel in Brno*

The Pisarky tunnel is the first motorway tunnel completed in the Czech Republic and the first tunnel excavated using the observational method. The variant consisting of a tunnel with relatively long cut-and-cover sections was chosen as the most considerate towards the surroundings.

Figure 10. The Pisarky tunnel, 1998.

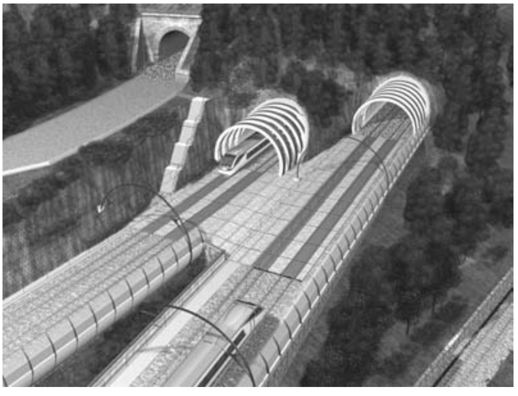

Figure 11. The new connection – visualization of the portal built in the centre of the city.

The tunnel is connected to a bridge over Certik stream on one side and to a bridge over the river Svratka on the other side.

4 RAILWAY TUNNELS

4.1 *The new connection in Prague*

Based on the EIA developed in 1996, it was decided that the railway lines from the Prague Libeǹ station and Prague Hlavni Nadrazi station and from Prague to Turnov would be led under Vitkov Hill, through a pair of double-rail tunnels. The reason for this solution was the effort to minimise the impact of railway traffic on inhabitants. Both 2400 m long double-rail tunnels were driven using the NATM.

Figure 12. The Vinohrady railway tunnels, 1944.

4.2 *The Vinohrady tunnels*

The year 1866 saw the commencement of the construction of the Franz Joseph's single-track railway line. As a part of this construction, the Vinohrady I single-rail tunnel construction started. The tunnel was later reconstructed and converted to a double-rail tunnel. The growth in the transportation outputs after the foundation of Czechoslovakia and the poor structural condition of the Vinohrady I tunnel resulted in the decision that a second double-rail tunnel (Vinohrady II) be built in 1940.

The recent reconstruction of the Prague railway junction which was carried out around the year 1985 required the addition of the Vinohrady III tunnel. It was driven using the ring method.

The Hlavni Nadrazi portal is located directly in the contact with the historic centre of Prague. We can say that it has become part of the unmistakable local colour of this area.

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Study of the ship lift structure at the Slapy Dam

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ABSTRACT: The paper deals with the design of the driven structures of the ship lift at the Slapy Dam, which is some 35 km upstream from Prague on the River Vltava. The dam profile is the main reason why the water route from Prague to České Budějovice is not navigable for ships of tonnage up to 300 tons. At present studies deal with several variants of the ship lift structure. Despite the fact that none of the variants has been chosen for further stages of the project preparation, the text describes in detail only one of the planned variants – the lock with high lift, which is the most interesting one from the point of view of tunnelling. This design of the ship lift uses to the maximum extent the newly built underground structures and at the same time it requires minimum work on the right bank. The excavation of the shaft and the driving will take place in the nearest surroundings of the dam and the hydraulic power plant which will require that extra safety and anti-vibration measures should be taken.

1 INTRODUCTION

The Vltava water route from Prague upstream ends at present below the Slapy Dam. The Slapy Dam is the only structure of the Vltava Cascade where the structure of the ship locking system is not finished. In 1955 the hydraulic structure was put into operation without the locking system and this situation still lasts.

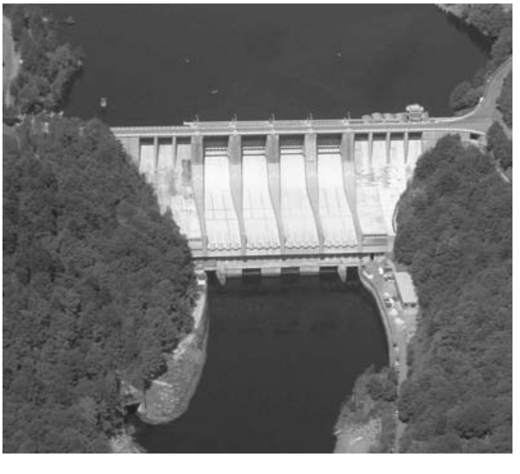

Figure 1. View of the dam downstream side. In the left-hand bottom corner you can see the diversion tunnel outlet.

When the Slapy hydraulic structure was under development, its first studies already planned that the locking system should be constructed. This goal was not met due to the lack of time, high building costs and technological difficulties in connection with the filling of the reservoir. Only the building part of the navigation lock on the right bank in the upstream pool was built.

The original project planned the ship lift structure for ships of tonnage up to 300 tons. A vertical ship lift in a reinforced-concrete tower structure on the right bank below the dam was planned, situated out-side the dam body. The structure should have been connected to the diversion tunnel, which was used during the construction activities for the purpose of diverting water from the dam construction site.

2 STRUCTURE HISTORY

2.1 *Dam*

The decision to build the Slapy Dam was made as early as 1933. The gorge in the place once called Midsummer Streams (Svatojánské proudy) was chosen as the most suitable one for the dam. The structure was approved in 1949. In the same year the work on the diversion tunnel and auxiliary structures was commenced. It was the first big structure of the Vltava Cascade after the World War II. The dam was built

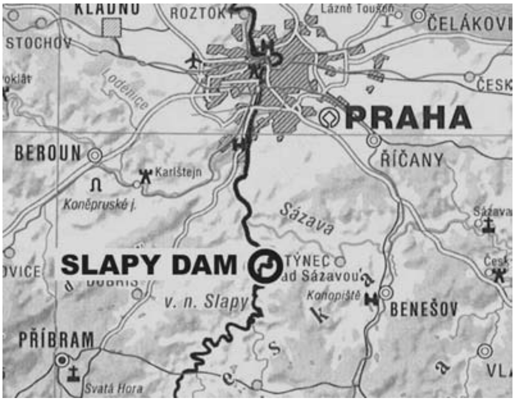

Figure 2. Map plotting.

in a dry pit, protected by fill dams on the upstream and downstream sides of the dam foundation surface. The dam was completed in 1955. The dam crest is 65 m above the bottom; the level difference is 54.6 m. The hydraulic power plant is situated in the concrete dam body. Water runs through four spillways with crest gates built above the hydraulic power plant.

2.2 *Diversion tunnel*

The diversion tunnel 10.0×12.0 m was built to allow rafts and vessels to navigate through while the construction activities were under way and most of all to divert water away from the construction site. The tunnel, 321 m long, was driven in a sound rock on the right bank and it is provided with concrete walling only in short sections by portals. The tunnel longitudinal slope is 0.15%. The diversion tunnel was closed shortly before the dam crest concreting. A concrete plug was constructed at the tunnel upstream end. It rested on cone-shaped pockets created in the wall and grouted with vertical holes, at the length of 30 m. Then, in April 1954, the filling of the reservoir was commenced. In July 1954 a vast flood came (a fifty-year one with the flow of 2200 $m^3.s^{-1}$), however the dam under construction withstood it. A positive result in favour of the capital of Prague downstream was the considerable transformation of the flood wave filling the empty reservoir.

At present the unsupported length of the tunnel is 291 m. The tunnel is accessible from the downstream water level (Fig. 3). The disposition of the tunnel and especially its length on the downstream side of the dam were designed from the beginning so that it could later be used as a navigation canal by the entrance to the locking system. The condition of the diversion tunnel was checked in detail some 20 years ago within trial blasting in connection with the shaft of the locking system. At that time the tunnel condition was quite good, only in a few places the rock under the vault toe.

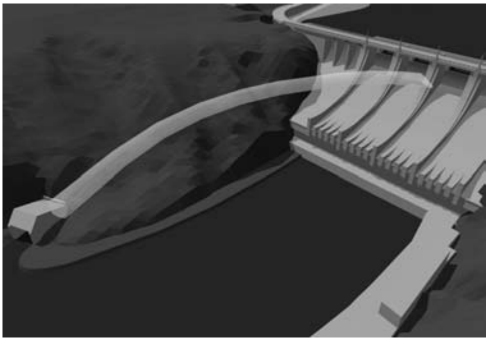

Figure 3. Existing diversion tunnel.

3 GEOLOGIC SITUATION

3.1 *Geologic conditions*

The Slapy hydraulic structure is situated in an area with monotonous occurrence of amphibolites of the argillaceous zone. The amphibolites are of the character of a fine-grained, solid, dark grey-green rock with high unconfined compression strength of up to 300 MPa. Occasionally in complete sections of amphibolites there is the so-called cushion lava forming longitudinally oval-shaped compressed schistose bodies, approx. 1.0×0.8 m. Amphibolites are very tough and resistant to chemical weathering, their faults are caused by slope movements of rock blocks and tectonic movements resulting from intensive compression of the argillaceous zone which is evident in schistose formations.

3.2 *Hydrogeologic conditions*

Groundwater absorbed in the rock base does not form a continuous horizon, its yield is small and the groundwater circulates in fissures without any relations. Apart from the surface sections, the fissures are close, frequently healed with calcite and almost water impermeable. The rock ambience permeability had been tested within research, before work on the dam was started, in holes by means of water-pressure tests and grouting tests. The value of the specific water loss was from 0.001 to 0.014 $l.s^{-1}.m^{-1}$ at the water pressure from 0.4 to 1.0 MPa. The dam profile was sealed with a grouted curtain during the construction activities. Since the dam was completed, the effect of the fissure systems pressure after afflux in the reservoir has not been tested.

4 SHIP LIFT CONCEPTION

4.1 *Original designs*

The heated discussions among experts in the Czech Republic, focused on the locking system structure at

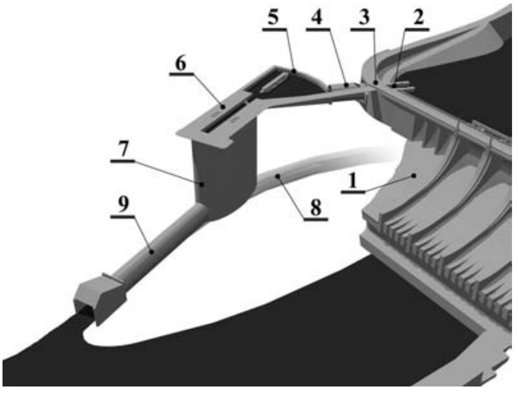

Figure 4. Design diagram. 1. Slapy dam, 2. Existing navigation lock, 3. Existing bridging, 4. New tunnel on the upstream water, 5. Passing bay, 6. Navigation lock platform, 7. Lock with high lift, 8. Closed part of the existing diversion tunnel, 9. Lower navigation tunnel, straightening the existing diversion tunnel.

Slapy, have lasted for more than 50 years. Up to the present several technological conceptions have been prepared (a lock with a high lift, a vertical ship lift in a shaft and a sloping ship lift). There were considered also alternatives of a navigation lock with s reversible pumps, a navigation lock with spare basins, a navigation lock with an ejection effect or a vertical ship lift with a complete balancing. Another proposal was an innovative design of a revolving ship lift when the ship channel moves along one carrying rail the shape of a helix Záruba (1960). Another original solution protected with a patent is a ship lift with pushed chains Záruba (1987).

4.2 *The existing state of project preparation*

The solution is designed in accordance with currently valid parameters of Water Routes European Classification Class I as regards ships of tonnage up to 300 tons. The ship lift must be able to lock a ship of effective dimensions 45.0×6.0 m. The navigation lock construction brings about a number of problems both hydraulic, nautical and static.

The last expert study (Uher and team 2006) contained three variants of the ship lift design: a shaft ship lift with a complete balancing, a lock with high lift with spare basins and a sloping ship lift.

The first and second variants have in essence the same layout, which is also used in all the previous designs. It uses as much as possible the existing navigation lock in the upstream water and part of the existing diversion tunnel for ships arriving to the lift on the downstream water.

The next part of the paper describes the most interesting variant from the point of view of tunnelling – the lock with high lift (Fig. 4). One of the problems of this design is the arrangement of structures with respect to ensuring optimum entry and exit conditions from upstream and downstream pools. In order that a ship enters and leaves on the downstream water it is necessary that this manoeuvre on a straight line. For this reason it is necessary to straighten the existing diversion tunnel in the point of the connection of the vertical shaft. The sudden change of the course of a ship which is necessary with respect to the above specified disposition will take place when the ship stops in the passing bay on the upstream water.

5 LOCKING SYSTEM STRUCTURAL DESIGN

5.1 *Upper navigation canal*

An upper navigation canal will be driven from the existing navigation lock, running partly through an open cut and partly through a short navigation tunnel. The new profile will be driven by means of NRTM (New Austrian Tunnelling Method) with a primary support of the excavation with a sprayed concrete layer and a system anchorage. Final walling will be of cast-in-situ reinforced concrete. On both sides of the tunnel will be (escape) footbridges, their vertical parts will be, within the fluctuation of water level of 1.50 m, used as guard rails.

The navigation tunnel will end in the passing bay tank of trapezoidal layout. The passing bay will be situated in an excavated pit, slopes of the cuttings will be supported with permanent anchors and protected with a sprayed concrete layer. The passing bay tank structure will be made of cast-in-situ reinforced concrete for water structures. The foundation surface of the passing bay bearing structure will be permanently drained with a drain system connected to the downstream water because of the pressure effects on the structure. The passing bay will have two mobile binding systems. With the systems it will be possible to turn the ships if they avoid each other in the passing bay.

In the downstream direction the passing bay tank is connected to the lock with high lift.

5.2 *Lock with high lift*

The navigation lock will be situated in a shaft of an elliptic layout 60.0×25.0 m, cross-section area is approx. 1300 m^2 and depth 60.0 m. The clearance of the lock is 45.0×6.0 m, the intermediate space will be used for spare basins (Fig. 6).

The shaft will be excavated gradually from the surface, it will be secured with primary walling of sprayed concrete with a system anchorage. To prevent the effect of shocks on the structures of the dam and the operated hydraulic power plant, before the excavation is commenced a secured perimeter nick will be

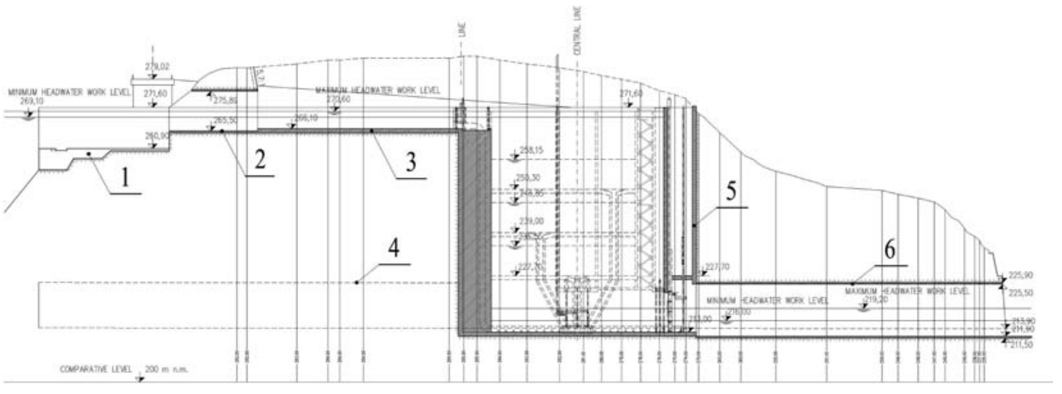

Figure 5. Longitudinal profile. 1. Existing navigation lock, 2. New tunnel on upstream water, 3. Passing bay, 4. Closed part of the existing diversion tunnel, 5. Lock with high lift, 6. Lower navigation tunnel, straightening the existing diversion tunnel.

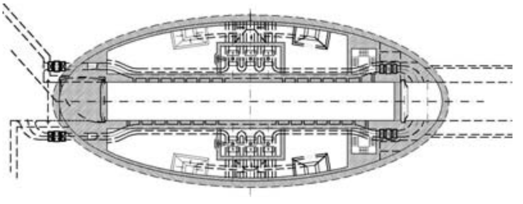

Figure 6. Plane section of lock with high lift.

made by means of a hydro cutter along the height of the shaft. The trench, 0.6 m wide, will be filled with bentonite mud. The final (secondary) walling will be of cast-in-situ reinforced concrete. Waterproofing with a permanent drain system connected to the downstream water will be inserted between the primary and secondary walling. This will eliminate effects of possible hydrostatic pressure from fissure systems on the final shaft structures. The inner clearance of the shaft enclosure will be approx. 60.0×25.0 m.

The shaft inner structures that will be used as spare basins are designed to transfer different hydrostatic pressures during regular water level fluctuation while the navigation lock is being filled and discharged. The lock will be filled and discharged by means of long bypasses closed with check gates. Bypass inlets on the upstream water will be on side walls between the tank and the lock. Check gates will be controlled with hydraulic face jacks. Shafts of bypass closures will be up to the level of the lock platform. In the downstream head the hydraulic face jacks of check gates of bypasses will be situated above the level of maximum downstream water. The navigation lock platform will be at 271.60 meters above sea level and the minimum water depth in the lock will be 3.0 m, the bottom at 213.00 meters above sea level. On the walls of the navigation lock will be installed floating binding elements for big and small ships and ladders. The lock platform will be lit.

The lock will have three twos of spare basins, interconnected with filling and discharging channels in the middle of the lock length with long bypasses. Particular channels will be closed with check gates controlled by hydraulic face jacks installed in the dry shaft integrated into the layout of spare basins. For the purposes of installing technology and its operation the shaft will be accessible from the lock platform. In the section of the channels between the check gates and the lock will be slots of box dam and the space will be filled with air. Particular levels of the spare basins will be connected to the atmosphere through the aeration pipe.

The head gate will be of boards with a vertical axis of rotation controlled by a hydraulic face jack. The gate sill with surface at 266.10 meters above sea level will be of minimum length, thus a ship hull will be prevented from sinking to the bottom when the water level drops while the lock is being emptied. In front of the gate will be the slot of the box dam for filled tube stop logs.

The tail gate situated outside the navigation lock layout will consist of a lift gate controlled by a hydraulic face jack installed in the dry shaft. The shaft floor will be at 227.70 meters above sea level, a box dam board will be permanently installed in the slots against downstream water. Access allowing technology installation and its operation will be possible from the lock platform top. The gate sill will be at 213.00 meters above sea level. The long bypasses will be emptied into the downstream water in the profile between the tail gate and the box dam.

For the purposes of operation and maintenance of the navigation lock and the spare basins there will be two dry access shafts with a staircase and a central

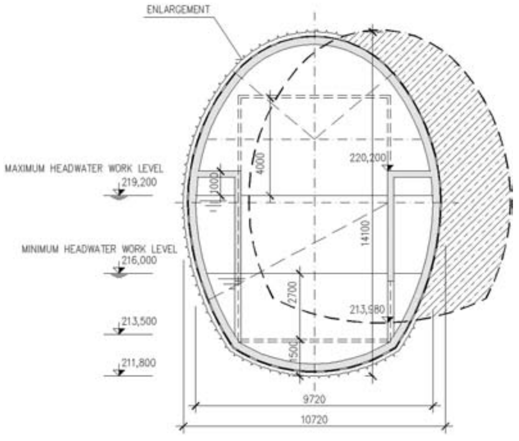

Figure 7. Cross-section of the lower navigation tunnel. Extension of the existing profile of excavation in the place of connection to the shaft.

space for material and equipment transport. Particular landings of the staircase will allow access to the floor of the machine room of tail gate closures, the machine rooms of closures of filling and discharging channels of spare basins and pressure covers provide access to particular spare basins. A small passenger lift will be designed to transport people.

5.3 *Lower navigation canal*

The lower navigation canal will consist of a tunnel running right in the direction of the longitudinal axis of the lock with high lift, connected to the straight end section of the diversion tunnel. The existing tunnel will have to be profiled anew (while using principles of NRTM with the primary support of the excavation with a sprayed concrete layer and a system anchorage). Final walling will be of cast-in-situ reinforced concrete (Fig. 7).

The remaining part of the diversion tunnel, approx. 130 m long, will be used for the fill of muck which will be subsequently grouted and the tunnel profile will be closed. When the operating level in the channel is at 216.00 meters above sea level, the minimum water depth will be 2.70 m (2.20 ship draught and 0.50 m margin). The tunnel bottom at the depth of 2.70 m will be min. 213.30 meters above sea level. Maximum level will be at 219.20 meters above sea level. On both sides of the tunnel will be (escape) footbridges with the top at 220.20 meters above sea level, their vertical parts will be, within the fluctuation of water level of 3.20 m, used as guard rails. The tunnel will be lit.

While the lower navigation tunnel is being driven and equipped, a pit will be excavated outside the tunnel portal in the River Vltava bed.

6 SAFETY ISSUES

6.1 *Safety measures taken during construction activities*

According to the task the work on the construction should be carried out while the hydraulic power plant works full out and the water level in the reservoir must not get lower.

The vertical shaft, some 60 m deep and some 1300 m^2 of excavated area, is a mining work of extraordinary dimensions. Excavation and driving will take place in relatively complicated hydrogeologic conditions (extremely hard amphibolites with compression strength of 300 MPa, broken into blocks with fissure groundwater under the pressure of up to 50 m of water column). Work will be carried out in the nearest surroundings of the dam and the hydraulic power plant, in the nearest place the shaft is 100 m away from the dam body. The establishment of the perimeter nick along the entire height of the shaft before the excavation is commenced is an extensive anti-shock measure taken for the purpose of separating the blasting operations from the outdoor environment. The said designs will be worked out in detail in the following stages of the project documentation.

6.2 *Safety measures during the ship lift operation*

As similar structure is found neither in the Czech Republic nor even abroad, the proposal of safety measures will be based on structures of a similar character, that means navigation locks with a big gradient.

If an accident of a ship in the lock occurs, the advantage of this variant is that it is possible to fill or discharge the lock as an emergency and thus the ship can be relatively quickly moved to one of utmost positions. For purposes of inspections and access of rescue teams there will be a section with ladders along the entire height of the navigation lock.

It is possible to anticipate that despite all the safety measures there will be a group of people that will refuse this way of transporting. In this instance these tourists will be transported by minibuses.

7 INCORPORATING THE WORK IN THE LANDSCAPE

7.1 *Effects on the environment*

The variant of the lock with high lift is to the maximum possible extent situated in the underground so that its adverse effect on the landscape is minimal (Fig. 8). In comparison with the other variants this one offers the slightest impact on the surroundings.

The lock with high lift construction is designed to meet the strictest ecological requirements as it is

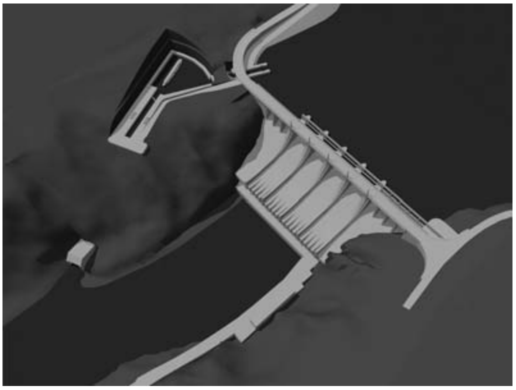

Figure 8. Perspective – bird's eye view.

situated in a beautiful landscape of the River Vltava valley, used as a holiday area by inhabitants of the capital which is some 35 km away.

7.2 *Ship lift as a draw for tourist*

It is expected that the entire ship lift system will be accessible tot the public. The variant of the lock with high lift is probably the least attractive from among the assessed variants if compared with the sloping ship lift when the ship is on the ground level all the time and thus the passengers can watch the valley. In order that the peace of mind of passengers is preserved the navigation lock and the lower navigation tunnel will be lit on walls. Within the reach of the work will be founded a centre to be visited by both ship passengers and the people who will only come to see the work. It is anticipated that the public will be interested in the work as it will be, owing to its extraordinary parameters and its design, a popular destination of tourists. This is also taken into account when the return on the investment is considered.

8 CONCLUSION

It is possible to say that the presented design prepared as a study is feasible on the basis of current conditions.

Safety risks in connection with the construction and operation of the locking system do not exceed common risks associated with the use of water routes and therefore they do not represent any obstacles to the construction.

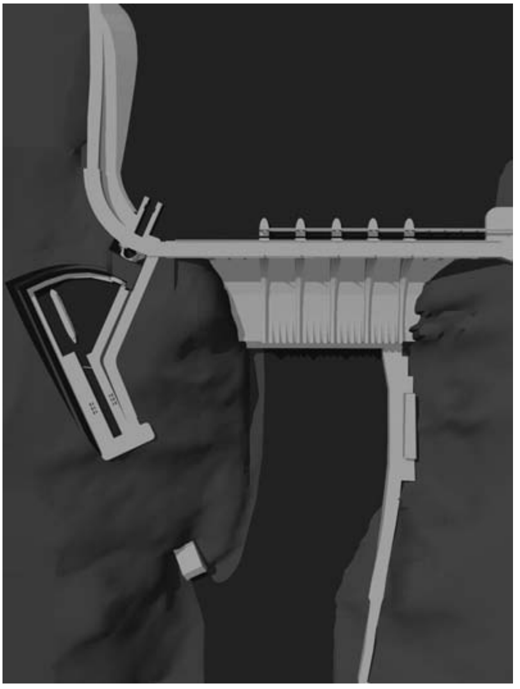

Figure 9. Incorporating the work in the landscape.

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Design of the District Heating Tunnel in Copenhagen

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ABSTRACT: The 3.9 km long District Heating Tunnel in Copenhagen is intended to supply the inner city of Copenhagen with district heating. Starting from a 37 m deep access shaft at the power plant on the island Amager, the tunnel is bored by an EPB shield machine and passes under the city to a 44 m deep shaft at Adelgade and a 30 m deep end shaft at Fredensgade. The tunnel has an internal diameter of 4.2 m and will carry 2 steam pipes, 2 hot water pipes and 2 condensation pipes. Under operation with 12 hours ventilation per day, the inside of the tunnel will heat up to a temperature of approx. 50 degrees. With its axis between 25 and 38 m below the ground surface, the tunnel is located at its full length within the Copenhagen limestone. The lining for this tunnel is designed with steel fibre reinforced concrete (SFRC) segments without conventional steel bar reinforcement based on the German SFRC design guideline. Apart from a general overview on the project, this paper is concerned with the methods and calculations applied in the structural design of the tunnel lining including the special aspect of the heating of the tunnel. Further attention is paid to the durability design, laboratory tests and experiences during construction.

1 INTRODUCTION

To upgrade the production and supply of district heating in Copenhagen it has been decided to move the current production from the two older plants Svanemølleværket and HC Ørestedsværket to the completely renovated block 1 at the Amager Power Plant (Amagerværket). In order to minimise the disturbance at the surface in the Copenhagen city it was decided to place the pipelines in a tunnel (Figure 1). The tunnel starts with a straight 2.4 km long stretch between the access shaft at the Amager Power Plant and the shaft at Adelgade, turns about 56° northwards and continues in a straight 1.5 km long stretch to the end shaft at

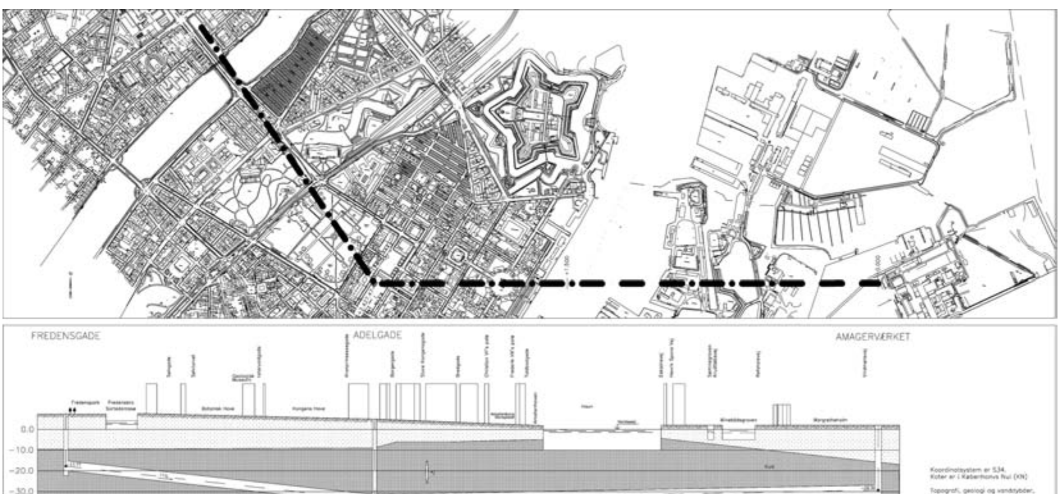

Figure 1. Layout of the District Heating Tunnel in Copenhagen.

Fredensgade. The tunnel is located at a depth between 25 and 38 m. The alignment is governed partly by the wish to excavate the tunnel primarily in the middle Copenhagen limestone and partly by minimum inclinations needed to deal with condensates in the steam pipes during operation. The heating tunnel passes below the future metro tunnel on the first stretch to the shaft at Adelgade. The geology along the alignment is characterized by quarternary layers of fill, clay and sand tills and melt water units down to depths between 14 and 18 m below the surface and Copenhagen limestone underneath (Knudsen et al. 1995, Hansen & Foged 2002, Jackson et al. 2004).

The shafts for the tunnel are constructed with secant pile walls in the upper part and are temporarily lined with shotcrete in the lower part. Permanent concrete linings with membranes at the outer faces will be constructed after the tunnel works are finished. The shaft at the Amager plant is the largest one. It has an oval cross-section with final internal diameters of 25/15 m and an excavation depth of 37 m.

The tunnel is excavated with an EPB shield machine. This type of TBM was also successfully applied in the Copenhagen metro project (Nymann & Taylor 2003). For the installation, launch, turning and arrival of the TBM, NATM caverns with temporary shotcrete linings are excavated from the three shafts. As the shafts, these caverns will also be lined with permanent concrete linings.

Although passing under densely built inner city areas with historical buildings and sensitive infrastructure, no surface settlement problems need to be investigated compared to shallow tunnels in soft soils (Kasper & Meschke 2006a, b) due to the deep location of the tunnel in the intact limestone rock mass. This conclusion is further supported by a comprehensive investigation of settlements experienced during construction of the Copenhagen metro phase $1 + 2A$.

The District Heating Tunnel is lined with steel fibre reinforced concrete (SFRC) segments without conventional steel bar reinforcement (Figure 2). Although the properties, design and possible application of SFRC have been a topic of research for a longer period (see e.g. Maidl 1995) only relatively few applications for precast segmental tunnel linings have been reported so far. To the authors' knowledge, the first large-scale test application of SFRC segmental linings without steel bar reinforcement was in 140 m of a pressurized water tunnel in the Lesotho Highlands Water Project in South Africa (Wallis 1995). Further application examples are the baggage handling tunnel at London Heathrow Airport (Moyson 1995) and, on a large scale, about 20 km twin-tube sections of the Channel Tunnel Rail Link (Davis et al. 2006). Design guidelines for SFRC have been worked out in various countries. However, no standards are available yet. The lining design for the

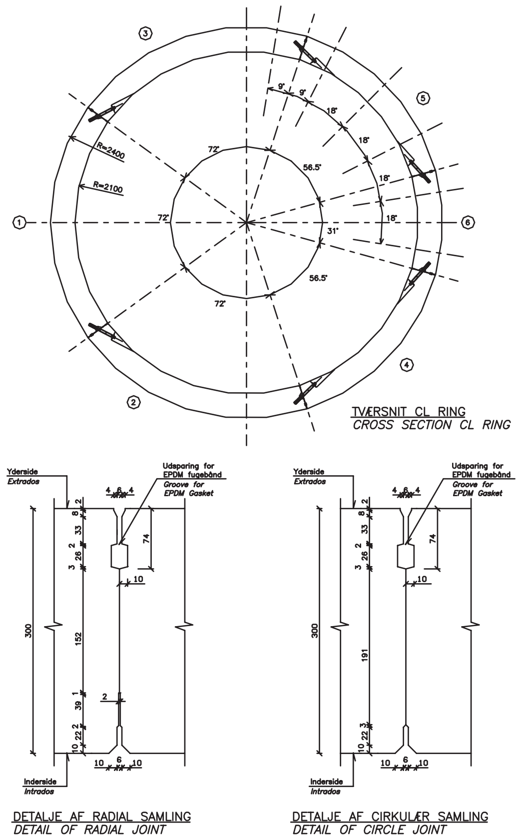

Figure 2. Layout of the segmental lining.

District Heating Tunnel is based on the German SFRC design guideline (DBBV 2001).

The annular gap around the lining is injected with a two-component grout (cement, bentonite, water and stabilizer as component A, water glass and water as component B), which is characterised by short gelling and setting time. This ensures a quick stabilization of the tunnel lining, avoids uplift of the rings and shear displacements between the segments.

The operation of the heating system has been optimized (Matlock et al. 2005). The tunnel is assumed ventilated 12 hours each day during night time leading to a predicted uniform temperature elevation along the circumference to max 54°C on the inside of the lining and to max 50°C on the outside of the lining.

The design and construction of the tunnel is performed based on a partnering model (late partnering) between the involved parties. The design work started in October 2004, the excavation works for the shafts started in September 2005 and the tunnel is due for operation of the pipelines end of March 2009. The total costs of the project are estimated to be DKK

750 million incl. the piping works tendered for in late 2006. The use of steel fibres was included as an option in the tender documents. Apart from direct commercial benefits to the contractor, it is anticipated that the client will gain from the use of SFRC by reduced operation and maintenance costs during the life time of the tunnel.

2 STRUCTURAL DESIGN OF THE SEGMENTAL TUNNEL LINING

The lining has a thickness of 30 cm (Figure 2). The concrete is C50/60 and contains 35 kg/m^3 fibres of type DUOLOC 47/0.80. According to its bending strength characteristics (cp. Figure 5), the SFRC is classified as fibre class 1.4/0.6. The lining has flat concrete-to-concrete radial joints and flat longitudinal joints with 0.9 mm thick bituminous packers. It is equipped with EPDM gaskets. The segments are designed to form tapered rings with a width of 1.50 m and a taper of 2 cm consisting of 3 standard segments, 2 counter key segments and 1 key segment. The final loading situation of the circular tunnel lining in the limestone rock mass is characterised by predominant compression with only small bending moments. Steel fibres in the concrete lining improve the resistance against spalling of corners and edges, introduce a ductile post-cracking behaviour, eliminate corrosion problems and therefore increase the durability. However, special care is required during segment production, transport and construction in the tunnel, as SFRC segments cannot compete with the bending capacity of conventionally reinforced segments. The segments are erected with a vacuum pad erector in combination with a shear key of 110 mm diameter in the centre of the segments. The segments are bolted together with straight bolts in bolt pockets. The bolts are removed after construction.

The sectional forces in the lining for the loading situation without heating of the tunnel have been determined by two-dimensional numerical models using the finite difference code FLAC (Figure 3). The model considers the different soil layers and the grout by continuum elements and the tunnel lining by beam elements. A fine grid is used in the vicinity of the tunnel. The lining is modelled in a simplified way as a continuous ring without consideration of the radial joints. This is a conservative approach leading to the largest possible bending moments while the normal forces in the ring are not affected by this assumption. Table 1 contains the material parameters applied in the analyses. Both an upper and a lower bound estimate of the limestone stiffness are investigated.

The calculated sectional forces contain a circle shape tolerance of the ring of 25 mm and lipping of 15 mm. The load bearing capacity of the lining is verified based on the stress-strain relationship shown in

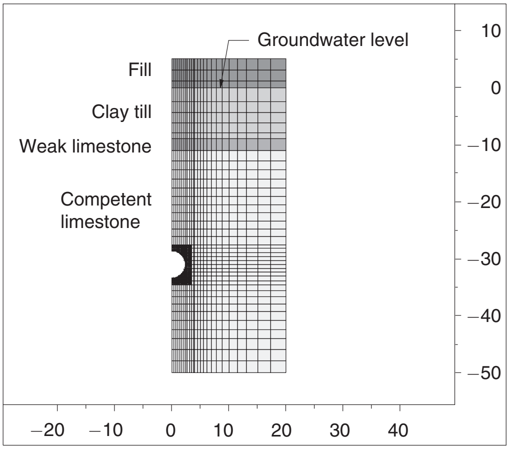

Figure 3. Numerical model for the determination of sectional forces without heating of the tunnel, shown for the deepest section.

Table 1. Geotechnical parameters of the different soil layers.

Material	γ (kN/m^3)	E (MPa)	ν –	ϕ' (°)	c' (kPa)	K_0 –
Fill	22	10	0.3	30	0	0.5
Clay till	23	150	0.3	34	10	0.44
Limestone crushed zone	22	1000	0.25	45	50	0.29
Competent limestone	22	1500 (min) 10000 (max)	0.25	45	100	0.29

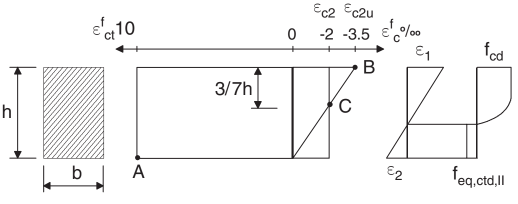

Figure 4. Rectangular SFRC cross-section: Definition of admissible strain ranges and assumed, simplified stress-strain relationship for the verification of the bearing capacity.

Figure 4. A parabolic-rectangular compressive stress block is assumed for the concrete and a simplified (conservative) rectangular tensile stress block is assumed for the cracked concrete. The actual tensile stress-strain relationship of the SFRC determined from bending tests is shown schematically in Figure 5. The resulting failure envelope in the M-N interaction diagram is shown in Figure 6 and Figure 7 together with the calculated sectional forces for the two limit cases

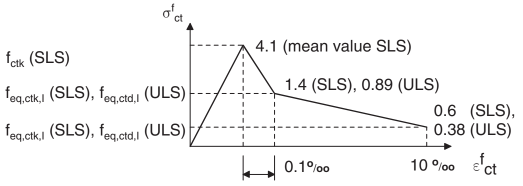

Figure 5. Tensile stress-strain relationship of the SFRC according to laboratory 4-point bending tests.

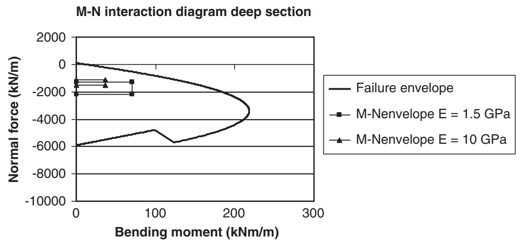

Figure 6. Failure envelope and envelope of the sectional forces for the deepest tunnel section without heating, for the upper and lower bound estimate of the limestone stiffness.

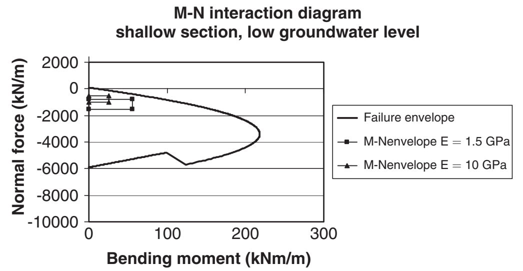

Figure 7. Failure envelope and envelope of the sectional forces for the shallowest tunnel section with low groundwater level, without heating, for the upper and lower bound estimate of the limestone stiffness.

of the deepest and the shallowest section of the tunnel. The ranges of the sectional forces are shown as rectangles, bounded by the lines min N, max N, min M = 0 and max M. The discontinuity of the failure envelope is due to the required higher safety factor of 1.8 for failure of uncracked cross-sections compared to the normal safety factor of 1.5 for failure of cracked cross-sections. It can be observed that without heating of the tunnel the smallest safety margin for the bearing capacity is obtained for the shallowest section with low groundwater level.

The heating of the tunnel leads to an axisymmetric temperature and, consequently, thermally induced strain and stress distribution and has therefore been analysed with an axisymmetric thermo-mechanical numerical model. The model contains the lining, the

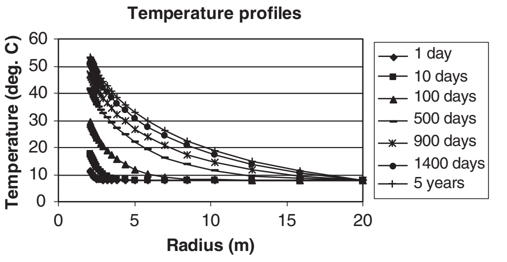

Figure 8. Computed axisymmetric temperature profiles in the lining, grout and limestone at different points in time. The steady state is reached after approximately 5 years.

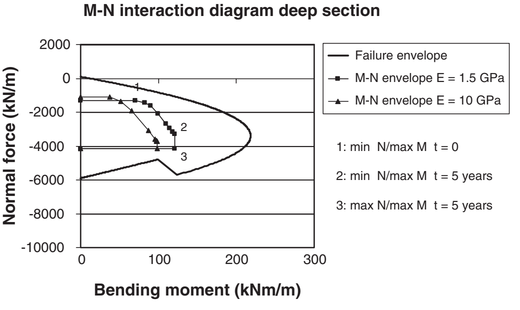

Figure 9. Failure envelope and envelope of the sectional forces for the deepest tunnel section including heating, for the upper and lower bound estimate of the limestone stiffness.

grout and the limestone and considers a constant heat flux on the inside of the lining of 19.7 W/m^2 in the SLS (totally 261 W/m, cp. Matlok et al. 2005). The initial temperature is assumed to be 8 °C in all materials and is assumed to remain unchanged at a distance of 20 m. For the mechanical plane-strain analysis, the displacements are fixed at a distance of 20 m. Figure 8 shows the evolution of the temperatures due to heating of the tunnel. The steady state is reached after approximately 5 years with a temperature of 53.3 °C on the inside of the lining and 49.4 °C on the outside of the lining. For the verification of the bearing capacity of the lining (ULS), a heat flux of $1.35 \times 19.7 = 26.6$ W/m^2 is applied resulting in steady-state temperatures of 66.1°C on the inside and 60.8°C on the outside of the lining.

The temperature induced sectional forces in the lining are obtained by integration of the computed tangential stresses and are added to the sectional forces without heating of the tunnel. The result is shown for the deepest section of the tunnel in Figure 9. The temperature increase causes a considerable increase of the normal force in the lining ring. Due to the temperature gradient over the lining thickness and the lipping tolerance the bending moment increases as well. Only the deepest section is shown as it is obvious that this gives the most critical sectional forces after heating.

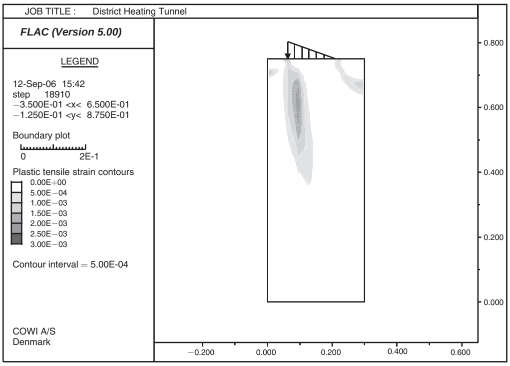

Figure 10. Computed plastic tensile strains (cracking) at the radial joints based on $f_{eq,ctd,I} = 0.89$ MPa (ULS).

The concentrated transfer of the sectional forces at the radial joints through the 152 mm wide contact area (cp. Figure 2) leads to bursting stresses in the vicinity of the joints. The most onerous sectional forces (M = 121 kNm/m and N = 4141 kN/m (ULS) and M = 90 kNm/m and N = 3074 kN/m (SLS)) have been transformed into an equivalent triangular contact pressure distribution and have been applied in a 2D-plane strain numerical model as shown in Figure 10.

Although the formation of a larger crack is observed in the ULS the predicted strain level in the crack of about 2 ‰ is small (cp. Figure 5). It is concluded that the cracks do not represent a critical failure state and that the integrity of the structure can be maintained by the steel fibres. In an SLS computation, no cracking is predicted as the bursting stresses remain below the mean characteristic tensile strength of 4.1 MPa.

3 DURABILITY DESIGN OF THE SEGMENTAL TUNNEL LINING

The client has required that the tunnel shall be designed to satisfy a service life of 100 years, where the structure should not require major maintenance and repair. The elevated temperature in combination with the aggressiveness of the environmental exposure requires special attention. The tunnel is located in salty water, mainly close to the harbour with chloride contents of 1–1.5%. Due to this, the most critical deterioration process in case of traditionally reinforced segments would be chloride induced reinforcement corrosion which, for structures in marine environment, would require special measures to guarantee a service life of 100 years (end of service life is usually defined by the start of reinforcement corrosion). In the present case, the situation is even more serious due to the increased temperature, as the temperature level is decisive for the rate of transporting aggressive substances such as chlorides into and within the concrete. All chemical and electro-chemical reactions are accelerated by an increase in temperature. According to a simple rule of thumb an increase in temperature by 10°C causes a doubling of the rate of reaction. In the present case the temperature is approx. 40°C higher than in usual, bored tunnels with approx. 10°C, which will lead to a $4 \times 2 = 8$ times faster ingress of chlorides in the heating tunnel compared to the ingress rate in normal, bored tunnels. In addition, the corrosion process may be accelerated due to the risk of accumulation of chlorides on the internal tunnel surfaces due to evaporative effects. These factors would make it very difficult to design the heating tunnel for a 100 years corrosion free service life when trusting alone on the traditional durability measures such as sufficiently large and sufficiently dense concrete layers. In the present case, the traditional design would require covers of approx. 70-80 mm. These covers would induce a high risk of spalling during handling and installation of the segments (ram thrust forces etc.).

Several research investigations (Raupach et al. 2004, Weydert et al. 1998) have shown a favourable behaviour of steel fibre reinforced concrete with respect to durability. This is due to the fact that the critical chloride content of steel fibre reinforced concrete (so-called threshold value) is much higher than that of normal steel reinforcement, although identical steel compositions are used. Further, the corrosion activity of steel fibre reinforced concrete is limited to surface rusting of some protruding single fibres. Although rust spots near the surface are expected to occur, corrosion will not penetrate deeply as a result of the following reasons in case of uncracked concrete:

- Continuous corrosion between individual fibres is not possible because each fibre is individually embedded in and thus protected by the concrete. Therefore, no electrical conduction occurs which would result in a continuous process (no macrocell corrosion possible).
- Corrosion of fibres will not cause the concrete to spall. The stresses caused by the expansion of the fibre diameter are negligible and do not lead to more damage.

In summary, steel fibre reinforcement instead of steel bar reinforcement has been found to be the best solution to ensure the required durability of the District Heating Tunnel.

4 LABORATORY TESTS AND EXPERIENCES DURING CONSTRUCTION

The following material tests are made:

- Petrography analyses on plan-sections of extracted cores to investigate the distribution of the fibres in

the hardened concrete. In the beginning, the distribution of the fibres was not always as homogenous as desirable with some higher amounts of fibres in the lower part of the cores. This concrete test has proven to be very useful and almost indispensable, at least during the pre-testing and starting phase of the production.
- Wash-out tests of fresh concrete to determine the amount of fibres. The intention was to find a simple test method which gives a second control of the actual fibre amount in addition to the automatic weighing determination at the mixing plant.
- 4-point bending tests to ensure the required strength properties.
- Splitting tests (Brazilian tests) as additional tensile strength tests.
- Unconfined compressive strength tests to ensure the specified C50/60 concrete class.

About 300 rings have been built so far. The design, the segment production and the construction works appear to be satisfactory as segment damages are limited to 15 visible cracks with wet spots and some irrelevant spalled edges at the bolt pockets and erector cone recesses. Most of the cracks have occurred at the corners of key segments. Only a small number of further minor water leakages have been detected.

5 CONCLUSIONS

The construction works for the District Heating Tunnel in Copenhagen comprise the excavation of three deep shafts, short NATM caverns and 3.9 km of TBM tunnelling in the Copenhagen limestone. Following previous applications in some tunnel projects, the lining for the bored tunnel is made of SFRC segments without conventional steel bar reinforcement. This eliminates the problem of corrosion, ensures the required durability and helps to achieve a cost-efficient segment production as well as an efficient construction with low segment damage rates.

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Excavation connecting the underground ventilation station and the underground expressway tunnels

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ABSTRACT: Construction of the Nakaochiai Ventilation Station requires the underground connection of the shaft structure and the tunnels. Due to schedule constraints, the entire 50 m-section is to be opened up at the same time, so several countermeasures, such as heavy soldier piles, wedge concrete and soil struts, are adopted for construction. The construction sequence is thoroughly examined and instrumentation of the structures is planned. This paper introduces one successful example of underground ventilation station construction, focusing particularly on the connection between the ventilation station and the tunnels.

1 INTRODUCTION

In recent years, traffic congestion in urban areas has become a major problem as the Japanese population becomes more and more concentrated in large cities. To alleviate traffic congestion, expressways have been constructed in the large cities. The construction of the expressway system in the Tokyo metropolitan area began in the 1960s, but the construction of circular expressways lags far behind the construction of radial expressways, as shown in Fig. 1. When expressway construction started in the 1960s, elevated expressways over the public right-of-way were built to expedite the construction schedule and to minimize the construction cost. However, due to environmental problems and overcrowded ground-level space, plans were made to construct the first underground expressway in Tokyo.

The Central Circular Shinjuku Route is an 11 km-long western section of the Central Circular Route and the first underground expressway in Tokyo, as shown in Fig. 2. The Shinjuku Route is designed as a tunnel structure under the Ring Road No.6, which has an average daily traffic of 18,000 vehicles. One of the problems of underground expressways is the ventilation of exhaust gas. Ventilation stations are being constructed at approximately every 1,500 m. Most of the tunnel sections are built using the shield method, but the ventilation stations are built using a cut-and-cover method above the tunnels. Therefore, connections are made underground between ventilation stations and expressway tunnels, and new techniques

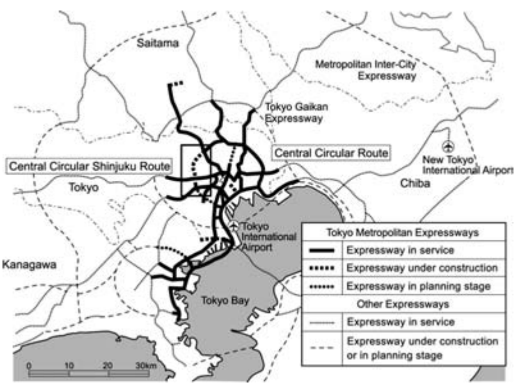


Figure 1. Greater Tokyo Expressway network.

will be developed to accommodate underground construction of expressways, such as underground tunnel enlargement and diversion.

2 DESIGN AND DETAILS OF THE NAKAOCHIAI VENTILATION STATION

The Nakaochiai Ventilation Station is 230 m long, 35 m wide and 22 m deep. The expressway road was built under the ventilation station using the shield method, so the excavation for connecting the ventilation station and the expressway roads reaches a depth of 36 m. Our contract concerns the southern 130 m of

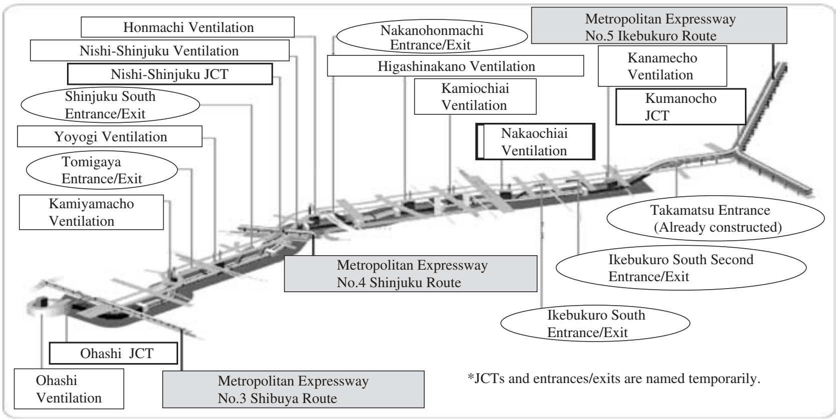


Figure 2. Over view of the Central Circular shinjuku.

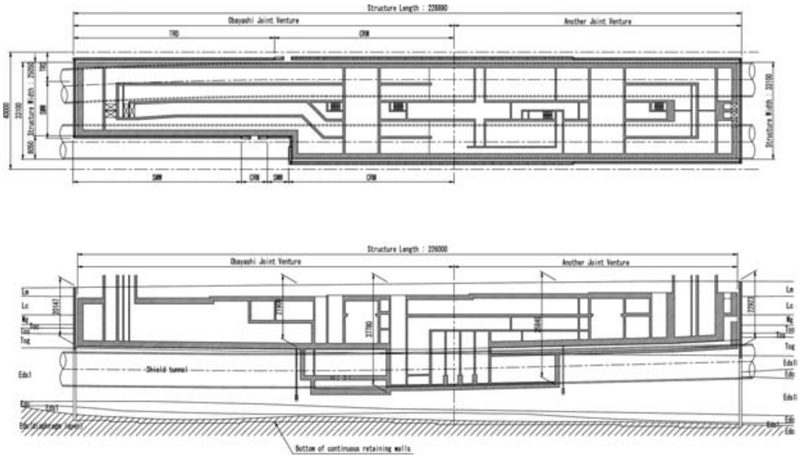


Figure 3. Plan view and longitudinal view of the ventilation station.

the station structure construction. The shaft, which will provide a connection to the tunnels, is 8.1 m wide, 50 m long and between 35 m to 37 m below the ground.

The ventilation station is built in stable diluvial strata. There is a layer of Kanto loam and clay from the surface to a depth of 10 m, followed by a layer of Musashino gravel and Tokyo gravel to a depth of 22 m. The ground underneath the Tokyo gravel layer is the 1st Edogawa sand layer, which is stable diluvial sand, and its N value is more than 100. The 1st Edogawa clay layer lies under the 1st Edogawa sand layer.

The underground water level is about 10m from the surface, which is approximately at the top of the Musashino gravel layer. The Tokyo gravel layer and the 1st Edogawa sand layer both have artesian ground water. When an earth retaining system was designed for the Nakaochiai Ventilation Station, a continuous retaining wall was adopted to prevent adverse effects on the surrounding area by dewatering. Therefore, the retaining wall extends to the 1st Edogawa clay layer, which is about 46 m below the surface.

The wall in the standard section consists of H-488×300 steel piles at 600 mm spacing. In the section where excavation is 36 m deep for connection to the tunnels, H-900×300 steel piles were used at 450 mm spacing, because no ground resistance was expected where the section between the two tunnels was excavated. The plan view and the longitudinal view of the ventilation station are shown in Fig. 3. The cross-sections are shown in Figs. 4 and 5. The Continuous Walls Using Recycled Mud (CRM) method, a kind of slurry wall method in which a horizontal multi-axis

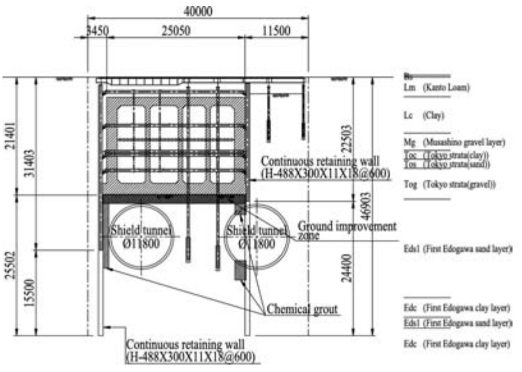


Figure 4. Cross-section of the standard section.

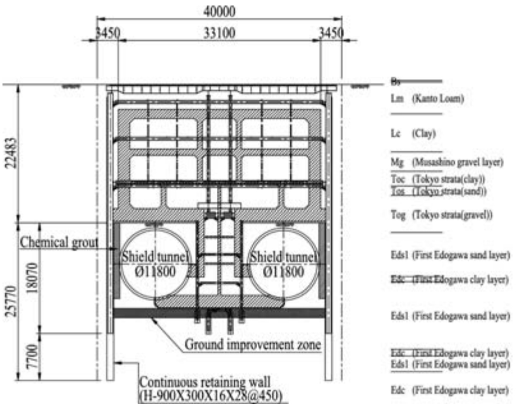


Figure 5. Cross-section of the connection to the tunnels.

Table 1. Quantities of the major work items.

Work items	Unit	Amount
Cotinuous retaining wall (SMW)	m^2	3,930
Cotinuous retaining wall (CRM)	m^2	5,590
Cotinuous retaining wall (TRD)	m^2	3,560
Intermediate piles	本	104
Metal decking	m^2	4,800
Earthwork	m^3	89,000
Trench timbering	t	1,890
Chemical grout	k?	1,120
Ground-improvement	m^3	4,900
Filling	m^3	3,600
Concrete	m^3	31,350
Reinforcement	t	2,760

cutting wheel excavator is used, was adopted for a slurry wall where H-900 × 300 steel piles were used. The size of the element, which is 1,100 mm wide and 3,200 mm long, is the same as for a conventional slurry wall. The quantities of the major work items are shown in Table 1.

3 FORESEEABLE DIFFICULTIES AND CONSTRUCTION SEQUENCE OF THE VENTILATION STATION

3.1 *Foreseeable problems with the ventilation station construction*

Due to schedule constraints, the entire ventilation station structure could not be completed before the shields came through, so the ventilation shaft and the connection work could begin only after the shields passed through the station area. The completion of the expressway has long been awaited and the opening date for the public has been set. Accordingly, every effort to expedite the construction schedule had to be implemented. The construction sequence was planned in order to minimize the work remaining after the shields passed through, and all necessary preparation work was planned as well as possible before the shields came through. Only excavation and concrete placement of the shaft and the connection area were planned after the shields passed through, and the ventilation station structure, shaft construction, and connection had to be conducted concurrently. Connection is usually performed by dividing the entire area into several sections and completing the excavation and concreting of each section before moving on to the next section. However, due to the schedule constraints, the entire length of the connection section between the shaft and the tunnels had to be excavated at the same time.

3.2 *Planned construction sequence*

The following measures were planned to expedite the completion of the entire project.

(1) The slurry wall was extended to the impermeable clay layer so that all shaft and connection work could be done in dry conditions.
(2) Heavy H piles (H900 × 300 × 16/28@450 mm) were used for soldier piles so that the retaining wall was strong enough to be supported between the base slab of the ventilation station structure and the ground underneath the tunnels. These heavy soldier piles were adopted so that no passive earth pressure inside the soldier piles between the base slab of the station structure and the ground under the tunnels was measured.
(3) The ground underneath the tunnels had to be consolidated by jet grouting to form soil struts.
(4) When the shaft was excavated to the top of the tunnels, the upper arches of the tunnels were secured at the top to prevent lateral movement of the tunnels and to secure the ground between tunnels and the base slab of the ventilation station structure.
(5) The shaft structure was concreted before excavation between the shaft and tunnels began.

(6) Though the segmental rings at the ventilation openings were cut, the main beams were not cut as long as possible. The connection of the segments and the shaft structure was designed as a pin connection. Stiffeners and gusset plates were welded to the main beams where they come into contact with the concrete to secure the bearing capacity, and the axial loads of the segmental rings were transferred to the shaft structure. The details of the connection are shown in Fig. 6.

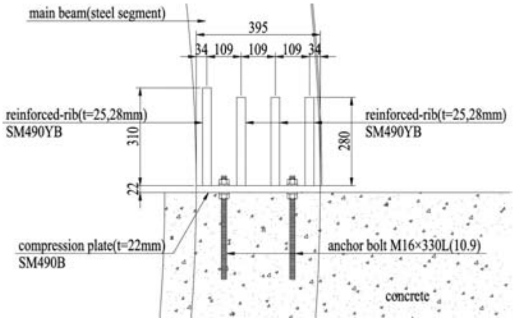


Figure 6. Details of the connection of the segment.

3.3 *The construction sequence*

The construction sequence is shown in Fig. 7.

Step 1: The struts for reinforcement of the tunnel linings were installed in the tunnels by tunnel contractors. The shaft was excavated approximately to the spring line of the tunnels and the pipe roofing was put into place. Steel pipes 87.6 mm and 114.3 mm in diameter were used for the pipe roofing. Then, the ground between

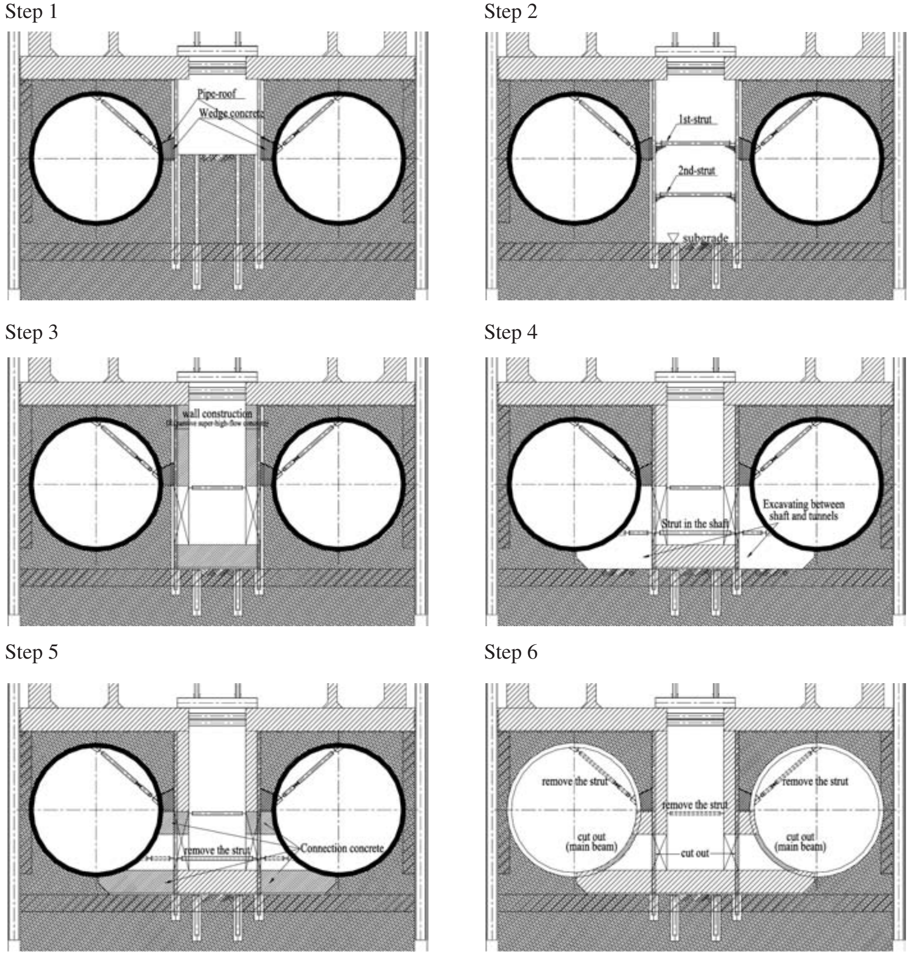


Figure 7. Construction sequence.

the shaft and the tunnels was excavated and the opening was filled with structural concrete, which prevented the lateral movement of the tunnels and secured the ground between the base slab of the station structure and the tunnels.

Step 2: The shaft was excavated to the bottom. Deformation of the soldier piles in the outside retaining wall was monitored by inclinometers, and the load cells were installed on the struts in the shaft excavation to monitor the axial loads.

Step 3: To secure the rigidity of the shaft structure, the concrete of the shaft structure was placed before the ground between the shaft and the tunnels was excavated.

Step 4: The ground between the shaft and the tunnels was excavated, while deformation and displacement of the tunnels and loads on the struts in the shaft was carefully monitored.

Step 5: When excavation was complete and the tunnels were exposed, the skin of the segments was cut out. The ribs and main beams were kept intact until the connection concrete was complete. The base slab of the connection was poured and then rest of the connection structure was completed. Bearing plates and gusset plates were welded to the main beams of the segments and embedded into the concrete.

Step 6: The main beams of the segments were cut out and struts inside the tunnels and the shaft were removed.

Since a wide area was open at the same time on both tunnels, deformation of the tunnels and stress of the shaft concrete and struts were measured. The following measurements were taken while the shaft and connection work was being conducted.

(1) Measurements of the tunnel's inner diameter
(2) Movement of the segmental lining
(3) Strain on the segmental lining
(4) Stress on the reinforcing bars of the connection structure
(5) Load on the struts inside the shaft

4 CONSTRUCTION RESULTS

4.1 *Installation of the pipe roof and placing the support concrete (wedge concrete)*

The pipe roof had to be put into place via the shaft, which was only 8 m wide, and excavation to the tunnels had to be performed by hand because of limited space.

Photo 1. Construction of wedge concrete.

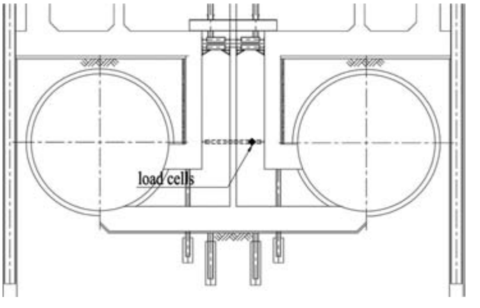


Figure 8. Location of the load cells on the struts.

The ground was hard, so the work was very time consuming and tough, as shown in Photo 1. However, this wedge concrete contributed significantly to the stability of the ground between the base slab of the station and the tunnels and to the stability of the segmental linings against lateral displacement. No detrimental movement was detected during the shaft construction, even though the entire 50 m-section was opened at the same time.

4.2 *Measurement of the load on the struts in the shaft*

After the shaft concrete was placed, the struts for the shaft excavation were replaced with new ones in the shaft structure and the axial load was monitored by load cells. The locations of the load cells are shown in Fig.8. Judging from the measurement results, which are shown in Fig.9, little change in the axial load on the struts was detected until the upper slab of the connection section was poured. It is believed that the expansion pressure of the non-shrink concrete, which was used on the upper slab due to its workability, may have caused the increase in axial load. The axial load far exceeds the design value, but it is still less than the maximum capacity of the concrete, and no major cracks were detected in the concrete either.

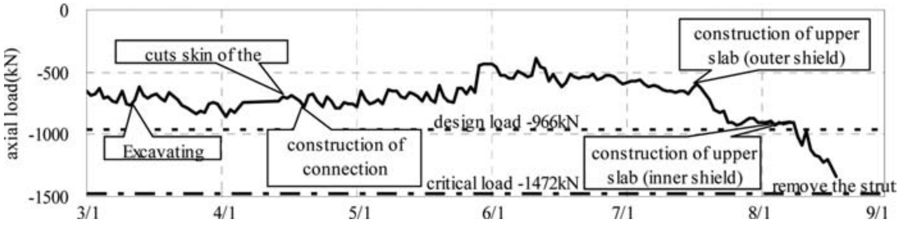
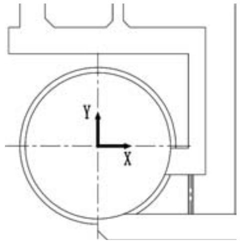
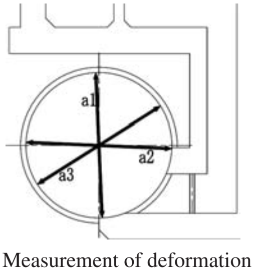


Figure 9. Measurement records of the load cells on the struts.



a) Measurement of displacement

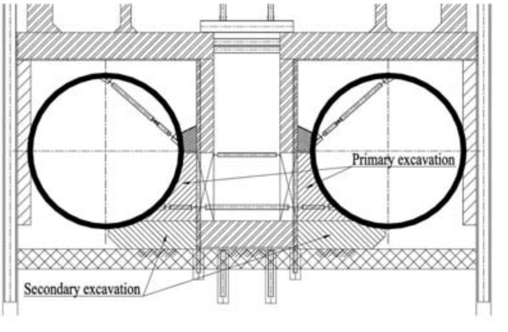


b) Measurement of deformation

Figure 10. Displacement and deformation measuring.

Table 2. Maximum displacement and deformation of the segmental linings.

Construction step	X (mm)	Y (mm)	a1 (mm)	a2 (mm)	a3 (mm)
Primary excavating	7	−3	−14	18	5
Secondary excavating	7	−13	−20	9	5



4.3 *Displacement and deformation of the segmental lining*

Once a day during excavation and once a week when concrete work was being conducted, the movement of the segments was measured using an automatic theodolite, while the deformation of the segmental lining was measured by an inner diameter-measuring device that uses lasers. The measurement locations are shown in Fig.10. The maximum deformation values in each stage are tabularized in Table 2. When the first stage of the excavation was completed, lateral movement was shown to have occurred, but the vertical movement was small because the bottom area of the segments had not been excavated. At the second stage of the excavation, the ground under the segments was excavated and 13 mm of settlement was recorded.

Photo 2. Excavation at the base slab of connection by a small backhoe.

Photo 3. Hand excavation under the segment.

Photo 4. Cutting out the skin of the segment.

As for the inside diameter of the segmental linings, the vertical diameter became smaller and the horizontal diameter grew larger as the excavation between the shaft and the tunnels proceeded. This tendency and the magnitude were within the expected range and no ill-effects were observed on the permanent structures.

Photos 2, 3 and 4 show the connection construction.

5 CONCLUSION

Due to the great challenge posed by the tight schedule, additional measures, such as installation of the wedge concrete and soil struts, were implemented. This was the first attempt to construct the entire 50 m section of the connection between the shields and the shaft at the same time. The field work was conducted carefully while measurements of the instrumentation were evaluated to ensure that the construction process proceeded as designed. Because the working area was relatively confined, work had to be performed by hand in many places, which could have resulted in schedule delays if it had not been planned in advance. The entire work of the shaft and the connection construction was completed as scheduled and without any major problems.

Construction plans vary according local geology and conditions, so foreseeable problems need to be investigated as much as possible in advance and necessary countermeasures must be prepared before construction starts.

Since underground connection work will become increasingly required in the future, this paper may help to assist in the planning of such work.

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Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina & Romancov (eds)
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Prague – Beroun, new railway connection

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ABSTRACT: A large investment, the modernisation of railway corridors, has been under way in the Czech Republic for approximately 10 years. The preparation of the modernisation of the Third Railway Corridor, leading from Prague through Plzen and Cheb to Germany, has progressed to an advanced stage. This corridor also includes the Prague – Beroun section. The existing route, which is 39 km long, currently leads through the valley of river Berounka. The design documentation for the reconstruction of this route following the existing route was prepared first. However, given the parameters required to be met by a design for a modernised railway corridor, the horizontal alignment of this route is very problematic in terms of the track speed, the direct interference with the Protected Landscape Area of Ceský Kras, and the problems stemming from the route passage through a densely populated area. This route modernisation would require large investment without achieving the expected improvement of the track parameters. This gave rise to the idea of a qualitatively different option of delineation of the route from Prague to Beroun. A new Prague – Beroun railway connection will be built to meet the high-speed track parameters, with the speed up to 300 km/h. Given the character of the landscape, the new route, with the total length of 28 km, mostly leads through a tunnel, approximately 24,7 km long, with the longitudinal gradients about 5‰.

1 THE EXISTING CONDITION

The route of the western part of the Transit Corridor III from Prague to Beroun, which is the route heading toward Plzeň and the border with Germany, leads along the densely developed Berounka River valley, and directly touches the Protected Landscape Area of Český Kras (Bohemian Karst).

The horizontal alignment contains sections with maximum design speed of 80 km/h. It was proven during the work on the draft design for the optimisation of this section that modification of the existing alignment was virtually unfeasible and, in addition, the investment would not result in the expected improvement of the parameters of the track. For those reasons the decision was made to seek a new solution that would follow parameters required for a high-speed railway track.

2 THE NEW DESIGN

A new solution to the western part of the Railway Transit Corridor III, the section Prague – Plzeň – state border, was approved by the Government in July 2005. This section is considered as a priority for it is part of the Corridor IV of the European network of main railway lines (AG-TC – C40), (AGTC – E40) Lvov – Čierná nad Tisou – Žilina – Ostrava – Olomouc – Prague – Plzeň – Frankfurt – Paris.

Given the terrain configuration, the newly designed route is 28 km-long, with a major part of its length (approximately 24.7 km) passing through a tunnel to be driven on a minimum longitudinal gradient of 5‰, under an overburden higher than 100 m. Obviously, given the anticipated length of the tunnels, the choice of dimensions of the tunnel clearance will also play an important role.

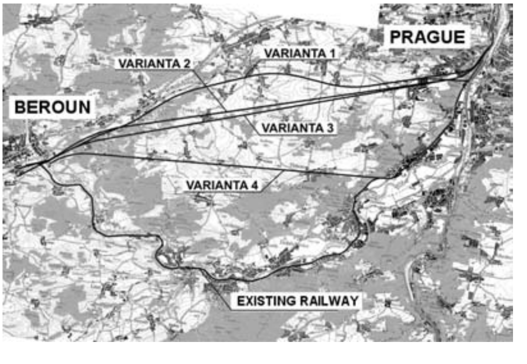


Figure 1. Variants of the route.

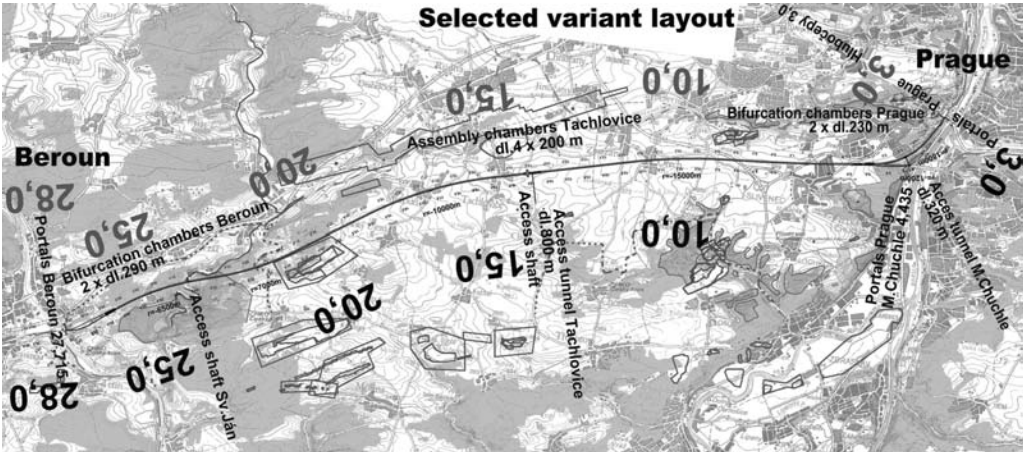


Figure 2. Selected variant layout.

The existing route, leading through the Berounka River valley, will remain without modernisation, only subjected to common maintenance, to serve the dense commuter traffic and as a by-pass route.

The studies which were submitted were assessed on the basis of a risk analysis. The most suitable solution was used as a basis for preparation of the design documentation for issuance of the zoning and planning decision, which is currently underway.

3 THE RISK ANALYSIS AND THE DOCUMENTED SELECTION OF VARIANTS

The risk analysis was carried out in the initial phase of the designing process. Naturally, it did not bring surprising results because the hazards defined at the beginning of the analysed progressions of segments of sources and concurrences are easily foreseeable even without any analysis. However, the risk analysis provides a consistent image of the entire design, not only of isolated parts. The particular risks, including their assessment, were processed in a tabular form.

The result of the risk analysis is a source of recommendations for the owner, designer and others, which are aimed at minimising the design risks. The recommendations cover the risk management, insurance, bank guarantees, dispute prevention and contract procurement.

As required by the order, the designer developed several variants of the possible design approach to the new railway link to be assessed. They consisted primarily of various variants of the alignment and variants of cross sections with various sizes and configurations. The route started at the Prague-Smíchov railway station, and ended in the Berounka River valley, on the edge of Beroun. The variants of the alignment differed in the end section found on the edge of Prague, in the location of the Prague portals, and in the vertical arrangement.

The alignment and configuration of the cross section which were selected on the basis of the risk analysis should meet the highest of all the requirements for basic design parameters, feasibility, safety and overall economy.

The most differing variants of the alignment are shown in Fig. 1; the selected variant is marked with No. 1. The following aspects were taken into consideration in the process of selection: the location of tunnel portals in relation to residential areas; conflicts of interest; passage through conservation areas; outlets to the surface in conservation areas; the possibility of achieving the design speed of 300 km/h; separation of the new high-speed railway line from the existing track; a necessity for modification of the existing track; a possibility for gradeseparated crossings of the two opposing traffic directions in the case of branching off; geotechnical conditions along the route in relation to the construction technique; problems associated with alteration of rock types; occurrence of karst phenomena and caverns during the course of the excavation; passage through a deposit area or its reservation; existing buildings above the tunnel; requirements for ventilation during the excavation and operation; feasibility of establishing site facilities; feasibility of the construction in terms of time; possibility of rail haulage of spoil; road haulage; spoil disposal; the tunnel length-cost relationship.

4 THE MAIN DESIGN PRINCIPLES

Apart from the basic order to connect Prague with Beroun by a double-rail line, the following main principles were sat out to be met:

4.1 *Feasibility*

The railway line will run through a pair of single-track tunnels, each 24.7 km long, according to the map No. 2 enclosed. The alignment must allow the travel at the speed of 300 km/h in the future.

The track will carry freight traffic and passenger traffic. For that reason the track must be split into two directions in Prague: passenger traffic to lead to the Smíchov station and freight traffic to the Krč station.

On the Beroun side, the possibility for the high-speed line to continue past the city as well as to branch off to the station in the city must be maintained. However, only the tunnels to the Beroun station will be built.

The horizontal alignment conditions do not allow branching off in the open-air track, the only branching possible is that in the tunnel.

The longitudinal gradient of the tunnel is 5‰ uphill from the portals to the mid point of the tunnel. The gradient changes at the grade-separated intersections.

The open-air route adjacent to the portals has not been designed to pass through residential areas so that the process of construction approval is not threatened by negotiations with the public, and a possibility of a conflict of interest with owners of existing properties is minimised. The areas for temporary site facilities were designed with the same attitude. The access to the tunnel is at chainages km 4.5 and 14.5 via two access adits and at chainages km 14.5 and 24.0 via two shafts.

The originally designed short at-grade section of the track near the village of Sv. Jan pod Skalou, which is also found in a conservation area, was also located underground as a result of forewarning of problems with the negotiation.

The tunnel will be driven by a TBM, with the sections between the portal and the branching off points excavated using traditional tunnelling methods. The profile of the tunnels is enlarged at the branching off locations and at the access adits connection points.

4.2 *Safety aspects*

The pair of single-track tunnels with separate portals is the basis of the safety concept. The tunnels are interconnected by cross passages every 400 m. This is how safe escape of persons from any point of the tunnel is ensured.

Dangerous contra-flow travels at the branching-off points have been eliminated by designing grade-separated intersections.

Access to the technical services centres are possible from the surface, therefore closing of the line in the tunnel to traffic is not necessary.

The ventilation system has been designed with respect to the needs for maintenance, contingent emergencies and a fire.

State-of-the-art equipment, which provides safety at the world's highest level, has been designed for the tunnels.

4.3 *Service life*

The structure has been designed for a minimum service life of 100 years. It will be solved so that optimum conditions for a long service life of the tunnel and equipment is secured in the way of which we have experience from Prague Metro. The tunnels and equipment rooms will be dry, ventilated and with relatively constant temperature.

4.4 *Environmental aspects*

The in the past completed tunnel structures have persuaded us that a continuous aquifer exists even in the relatively impervious Ordovician shales of the Prague Basin, up to the level of the tunnel at a depth of 100 m. Similarly, it was determined by measurements that the water table returned back nearly to the original level when the construction of a perfectly watertight tunnel had been finished. For that reason a waterproof lining around the entire circumference, the so-called 'submarine' type, has been designed for the tunnels on the new railway link between Prague and Beroun. This design, which does not require removal of groundwater by underground drains, eliminates permanent decline of water table and origination of dead wells or dry woods. It is very environmentally friendly a solution, which guarantees that the natural water regime will be restored.

5 ECONOMIC TUNNEL CROSS-SECTION AND SINGLE-TRACK TUNNELS THROUGHOUT THE ROUTE

The completed studies took into consideration variants with variously differing cross section sizes. The difference between the economic cross-section area and the large area which is assumed in German Railways' regulations amounts to 25%, which means a difference of about 6.5 billion CZK in terms of the cost.

The opposing traffic flows have been thoroughly segregated owing to the grade-separated design of crossings with the branching off tracks. The unfavourable effect of the pressure wave on passengers in common trains travelling in opposing directions will not, therefore, be experienced. The internal space of high-speed trains is sealed so that passengers are protected against these unfavourable effects.

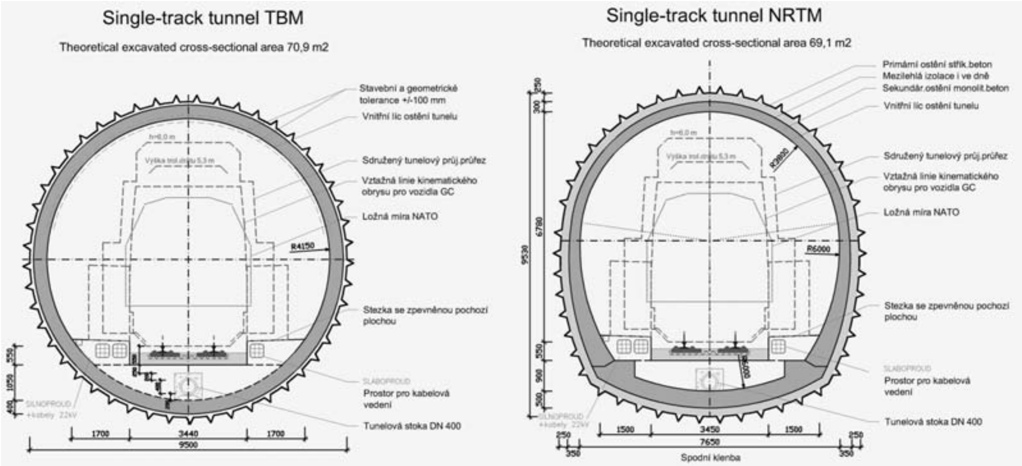


Figure 3. The typical driven tunnel cross-sections.

The above-mentioned measure allowed the owner to choose the economic cross-section. At the same time, the owner decided that the two tunnel tubes would be constructed simultaneously. The fact that a complete double-track railway line will be operated from the very beginning is a great benefit for quality and safety of the traffic. Regarding the environmental impact, the construction site surroundings will be inconvenienced for as short time as possible. Naturally, the total project cost is lowest when the construction programme is not divided into stages.

6 GEOLOGICAL CONDITIONS

Geological conditions are one of the main factors in terms of selection of the alignment. The tunnel route keeps clear of areas with mineral deposits and undermined areas.

The railway line passes through the Early Palaeozoic area forming the central part of the Barrandiene Member. The Palaeozoic rock layers rest unconformably on the basement formed by Proterozoic rock, which was folded during the Cadomian orogeny. The overall knowledge of this area is high, but, for example, a state-of-the-art survey of the tectonic pattern is missing and information from great depths is scarce.

The core of the Barrandiene Basin in Prague consists of a sequence of strata of Ordovician and Devonian sediments and volcanites. The rocks were folded during the Varisan orogeny and disturbed by transverse and longitudinal fault tectonics.

The western part of the tunnel passes through Ordovician rock, i.e. silty shale, siltstone and through quartzite of the Kosov Member, and Silurian rock types. The Silurian rock types are represented by graptolitic shale and marl slate. Shale and claystone prevail and frequent diabase intercalations occur in the lower part of the Silurian strata. The Upper Silurian rock mass contains larger proportion of limestone. It is characterised by a high degree of facies variability, frequent alternation of shale, various types of limestone and volcanic products disturbed by thrust faults.

The carbonate evolution following from the Silurian basement prevails in the Devonian strata in the eastern part of the tunnel. The Lochkov Member contains the Radotín and Kotýz limestone facies. A limestone sequence (Loděnice, Slivenec, Řeporyje, Koněprusy) can be distinguished in the Prague Member. The Zlíchov Member consists of grey, bedded limestone with hornstone. Significant karstification must be expected in the Devonian limestone. The current degree of knowledge obtained by surveys allows us to determine the border of possible karstification in the map with an accuracy of about 100 m. This border runs from the southern edge of Prague-Radlice through Ořech, past the southern edge of Tachlovice to Sv. Jan pod Skalou. According to the results of an information search based on long-term karstological surveys of the Bohemian Karst, it is impossible to determine the depth of a zone which would be beyond the karstification reach. Conversely, manifestations of karstification are expectable throughout the depth of the limestone bodies, which are affected by folding to the depth of about 100 m. It has been ascertained that the occurrence of karst phenomena, tapping of karst cavities, water irruptions etc. are a source of serious risks and threats for mechanical excavation using TBMs.

North of this border, the possibility of occurrence of karst cavities can be completely excluded. The Dalejské Valley Shale is characterised by prevalence of greenish, in higher levels even reddish colour. The Srbsko Member consists of siltstone and shale. The thickness may exceed 250 m in synclines.

7 THE EXCAVATION METHOD

7.1 *TBM drives*

A major part of the tunnel, from the bifurcation chambers in Prague to Beroun, will be driven by a large-profile TBM. Most long tunnels in the world have been driven by TBMs. It is so for the following reasons:

Faster advance rate compared to traditional methods. The shorter construction time means less uncertainties for the owner and the tunnel is cheaper.

The equipment is ever more perfect after the numbers of tunnels driven mechanically in the world. They are capable of coping with mixed geological conditions.

Mechanical excavation is safer; workers do not get into contact with rock and cannot be injured by collapsing rock mass.

Mechanical excavation allows accurate control over the excavated volume; at the same time, it does not disturb the surroundings of the excavation by blasting operations, thus overexcavation is avoided, cavities do not originate behind the lining and settlement of the surface is minimised. This is advantageous when residential areas are being passed under.

The circular profile of the tunnel is the most advantageous in terms of structural analysis, thus the safest at an adequate cost.

With respect to the anticipation that stretches with relatively low hardness of the rock mass will be passed through, the TBM type which has been chosen is not equipped with grippers. This means that the TBM-driven tunnel will have a single-shell segmental lining consisting of reinforced concrete segments, which will provide support to the thrust cylinders propelling the TBM. The lining segments will be made of water retaining concrete; they will be provided with sealing gaskets on all sides, thus the lining will be waterproof.

7.2 *Traditional NATM excavation*

The NATM will be utilised for the excavation of the single-track tunnels which starting at the Prague portals in Hlubočepy and Malá Chuchle and continue up to the bifurcation chambers, for the bifurcation chambers at Prague and Beroun, the access adits and shafts, assembly chambers, cross passages and equipment rooms. There are the following explicit reasons for this decision:

Frequent changes in the shape of the cross section (each of the above-mentioned openings has a different shape).

The NATM-excavated parts are enabling works for preparation of the TBM sets and access adits and shafts.

The majority of karst phenomena which will be encountered in the Prague area are much easier for the NATM to overcome. The access adit in Malá Chuchle or other works may even be utilised for geological exploration purposes.

The NATM excavated works, e.g. the access adits and assembly chambers, can be carried out in advance of the TBM excavation or concurrently with it; it will result in reduction of the construction time.

The NATM-excavated tunnel lining structure will consist of a sprayed concrete primary liner, intermediate waterproofing and cast-in-situ concrete secondary liner. The waterproofing will be of the 'submarine' closed type, covering the entire circumference including the bottom. The secondary lining will have an invert structure throughout the NATM-excavated tunnel length to resist the hydrostatic pressure.

8 THE CONSTRUCTION PROCEDURE AND LOGISTICS

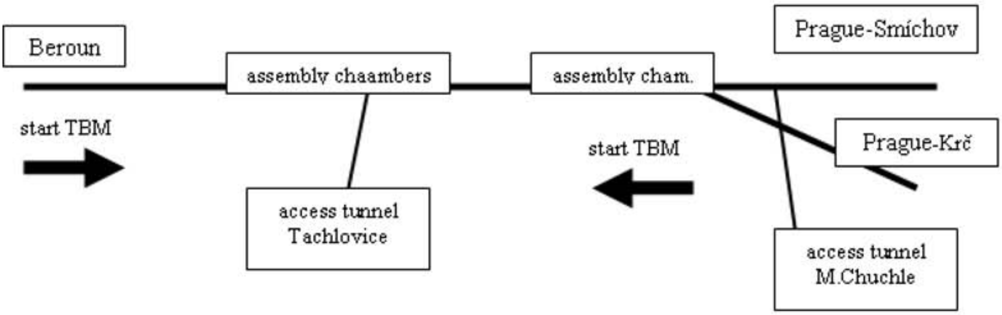
A single TBM would need approximately seven years to drive the about 22 km-long tunnel. This is a too distant horizon. Tunnel cost analyses have shown that over a half of the total cost consists of components dependent on time. The total construction time is therefore a deciding factor. For that reason, two TBMs must work concurrently on the drive of one single-track tunnel tube.

Problems may be encountered during the course of the simultaneous excavation by two TBMs when geological weakness zones are being passed through, which may result in suspension of the drive so that measures improving the rock mass can be implemented. In such a case, the work is much simpler and faster if an auxiliary adit can be driven from one tunnel to the other and the operation be carried out from this adit. Such the situation has already taken place in the world and examples of solutions are known. This is the reason why simultaneous drives of both single-track tunnels must be recommended. This recommendation is supported by the results of the risk analysis.

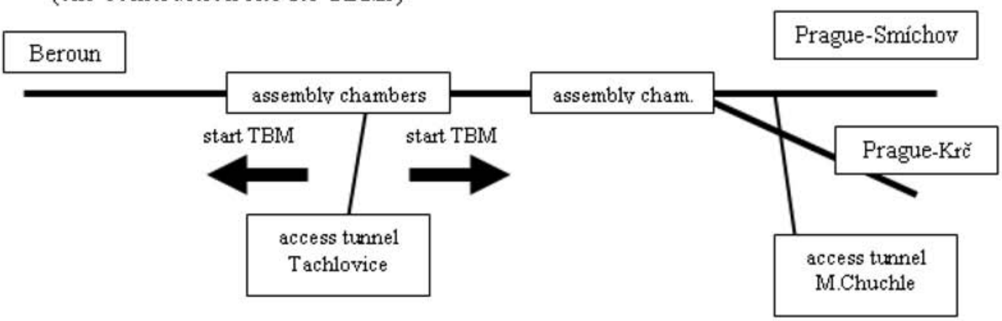
The excavation procedures possible are presented in the following chart. The duration of the tunnel drives is, according to the programme, about 4 years, the total construction time is 6.5 years.

In the case of volumes of material transfer during excavation of tunnels of so great length there

1. **heading from both portals simultaneously** (logistics cannot be provided on the Prague side)



2. **heading to both sides from the access adit connecting in the middle of the tunnel** (one construction site for TBMs)



3. **heading from the portal and from the access adit in the middle of the tunnel**, in the identical direction

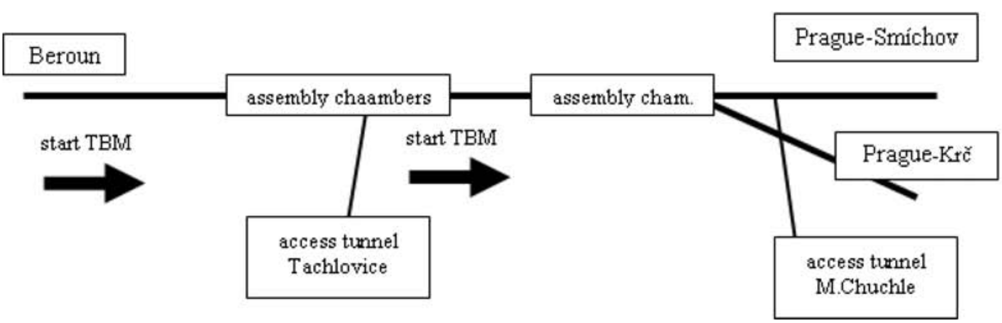


Figure 4. TBM excavation procedures.

is a significant problem associated with logistics, prefabrication and transport of the lining segments, and, primarily, haulage of muck. The design of the areas in front of the access adits and tunnel portals which are necessary for intermediate stockpiles of muck and intermediate storage of the segments allows for the needs; it has been proven that the project is feasible.

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Underground high-speed railway – The phenomenon of urbanistic development in Prague

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ABSTRACT: The Prague subway clearly demonstrates how important the utilization of the underground is for city life. A survey of the development of views about how to organize the basic pillar of the public transportation system. Realization of individual lines and subway stations in the historic section of Prague, in the central city, in the surrounding quarters and in the new suburbs. The transformation of conceptions concerning urban planning from the dominance of automobile traffic to the preference of walk-only zones in city centres. The subway in Prague is one of the crucial factors of urban life and development.

1 INTRODUCTION

After more than thirty years of operation of subway in Prague, we can consider as proven that the underground high-speed railway has besides the basic transport effect also urbanistic significance – city-making. I am going to prove this thesis on an example of several subway stations. Prague has unique specifications in the terrain configuration, geological conditions, but mainly in cultural and historical value of its historical centre, whose total area is almost 900 ha. It has been declared by UNESCO organization as the world cultural heritage for its thousand year's continuity, exceptional well-preservance and integrity and quality of monument sets. This heritage binds for systematic caution and modesty when executing any construction activity and therefore its makers have to deal not only with technical problems.

2 DEVELOPMENT OF OPINIONS ON THE CONCEPTION OF HIGH-SPEED RAILWAY

At the end of 19th century requirements on the transport were increasing with the development of housing. An extensive clearance of the Old Town and Josefov – the Jewish town was carried out, as well as the construction of infrastructure, embankment streets and walls regulating the flow of the Vltava River. Omnipresent construction activity inspired in 1898 Ladislav Rott to propose to use the wide trenches and demolitions for the construction of underground railway (Fig. 1).

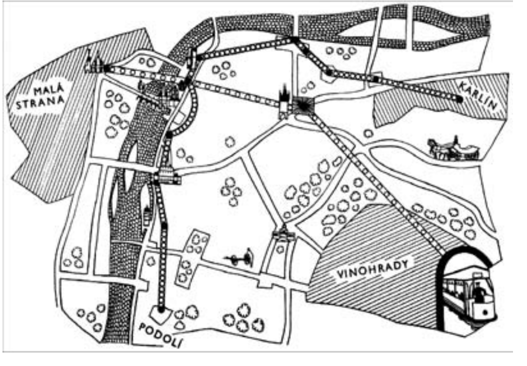

Figure 1. The scheme of Ladislav Rott proposal from 1898.

City technical officer Bohuslav Vondráček proposed in 1912 underground tram railway between Jindřišská str. to the Wenceslas square to Kaprova street. The number of citizens was close to 700 000 after the creation of Czechoslovakia in 1918 and Big Prague in 1922. Jiří Hruša published a proposal in July 1926 to solve the underground high-speed railway by placing it to deeply based tunnels with 3 diametrical lines through the city centre (Fig. 2).

First technically elaborated and complex proposal for the subway system in Prague was a study proposed by *Vladimír List* and *Bohumil Belada* from December 1926, which counted with 4 underground lines (Fig. 3).

A *competition* was announced for the solution of transport in the Big Prague in the year 1930. Results of this competition proved the necessity to introduce

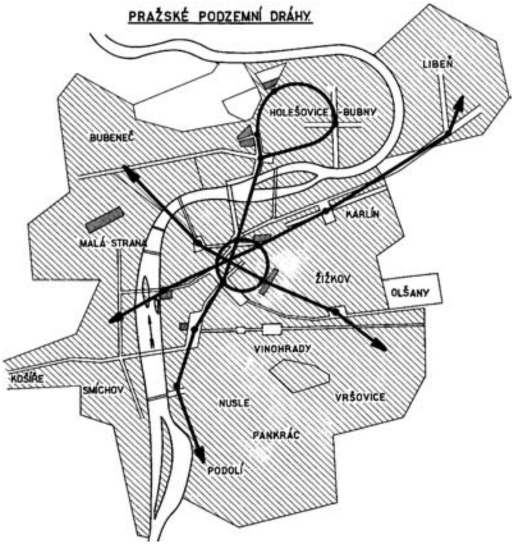

Figure 2. The proposal of Jiří Hruša from 1925.

Figure 3. The proposal of Vladimír List and Bohumil Belada from 1926.

a bearing element into the public transport into the underground.

The study department of municipal Electric companies elaborated in 1938 so called *Project D*, according to which the lines of dominant transport directions through the city centre were supposed to be carried out by cut-and-covered tunnels (Fig. 4).

Project D was rejected and *Konsorcium* of all considered partners for the construction was created. They proposed in 1941 the *Project M* of a classical subway underground railway. They supposed a system with 3 underground lines carried mainly shallowly under the ground with an exception of overcoming the Vltava River. Stations were supposed to be also cut-and-covered with lateral or insular platforms, underground halls and underpasses (Fig. 5).

It was even supposed after the Liberation in years 1946–1947 that it would be possible to begin with the subway construction in Prague according to the Project M, but it was not possible.

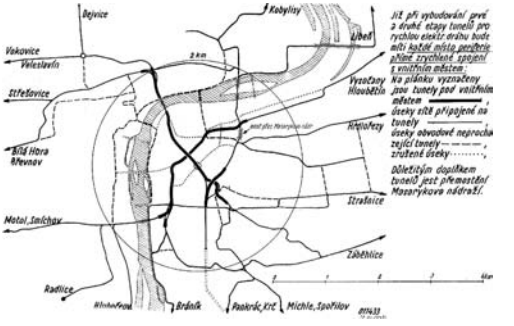

Figure 4. Project “D” from 1939.

Figure 5. Project “M” from 1941.

In 50’s when the number of inhabitants exceeded 1 million, were elaborated several proposals of territorial plans and also several studies on the solution of subway situation, but they have not succeeded to pass the *Directional master plan* until 1964, whose part was also the solution of public transport with a system of 3 underground lines in the city centre. The system called *Underground tram* (*PPT*). It required extensive groundwork in confined conditions of medieval streets, demolition of historical monument buildings, which evoked big conflicts with city infrastructure. The construction of PPT begun in January 1967 in Opletalova Street, close to the Central Railway Station (Fig. 6).

International expertise rejected project PPT in 1967 and the government then decided that a *classical subway* is going to be constructed in Prague. Projects were reelaborated and the construction was without interruption reoriented from the cut-and- covered tunnels to the bigger proportion of driven works.

3 CURRENT SITUATION IN THE SUBWAY NETWORK IN PRAGUE

There is a majority of railway tunnels between the stations with single or twin-track with stations in the

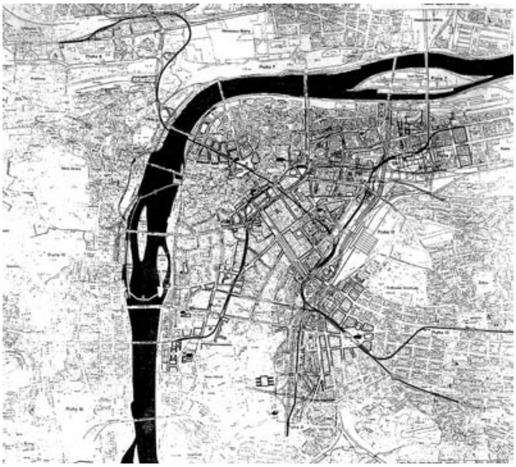

Figure 6. Project of the underground tram from 1964.

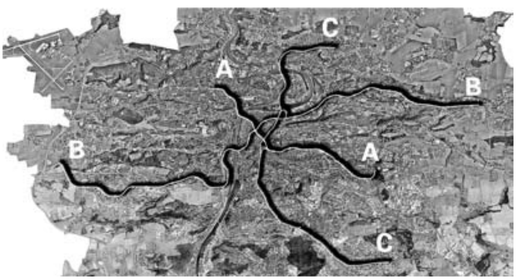

Figure 7. Current situation of Prague subway network.

historical centre and inner town driven and in the outer town cut-and-covered from the surface. All stations except for one are cut-and-covered or surface on the line C (Fig. 7).

An overview of Prague subway network.

Line	Construction length	Stations			Total
		Driven	c. and c.	Surface	
A	10,79	9	3	1	13
B	26,399	13	8	3	24
C	18,77	1	15	1	17
Total	55,959*	23	26	5	54
%		45,6	48,1	9,3	

*from which: 41,5 km driven tunnels = 74%,
13,5 km cut-and-covered tunnels = 24,1%,
1,6 km surface track = 1,9%.

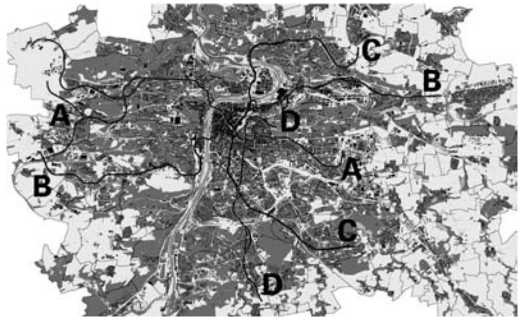

Figure 8. Prague subway network with development intentions.

4 DEVELOPMENT OF SUBWAY NETWORK IN PRAGUE

Subway in Prague is considered as a priority in solving the complete transport problems, because it currently realizes up to 40 % of all transport requirements for the public transport. This proportion is going to even increase by broadening the network. Strengthening the performance of subway is for Prague one of the most effective weapons against clogging of the city areas by car transport and against restricting pedestrian transport mainly in the city centre.

On the *northern end of C line* begun construction of 3 new stations, which are going to be opened in 2008. The construction of *a new 4th line D* is being prepared, from the Central Railway Station through Vršovice, Nusle, Pankrác to Krč, Lhotka and Písnice with the change to all three lines and with extension to the west to Žižkov.

The *extension of line A* westwards should be another extension to the subway network from Dejvice through Veleslavín, Petřiny to the hospital in Motol, to Bílá hora and from here to the airport Ruzyne and with a branch to Řepy and Zličín.

There is prepared turning of the branch on the East from the station Strašnická to Zahradní město and Hostivař and a branch from the station Skalka further on to the industrial area can be executed also (Fig. 8).

The system of Prague subway is fully opened and it allows anywhere and anytime to connect to the current system and to increase the quality of transport service in the city.

5 PRAGUE SUBWAY IN THE HISTORICAL CENTRE

The centre of Prague had a determining influence on the arrangement of subway network and its space solution. Saving majority of railway, stational and escalator parts of tunnels to the underground enables to limit the intervention to the medieval city only to entrance halls and underpasses. In the area of historical centre is altogether 10 stations, 8 of which are driven and 2 cut-and-covered and 14 shallowly based halls.

Both shallowly based stations on C line, Hlavní nádraží and Muzeum were built in 1974 including their halls on unfinished buildings according to the PPT project.

Hlavní nádraží station – integral part of underground foreground of central railway station, initialized its reconstruction and its finishing to its current appearance (Fig. 9).

A component of the station *Muzeum* on C and A lines is a wide underpass, which provides not only the entrance to both the crossroad station on C and A lines, but also an important connection for the city between Wenceslas square and the area of Vinohrady with the National Museum (Fig. 10).

Malostranská station on A line is the foreground of unique area of Malá Strana with beautiful palaces, gardens and town houses in the area under the Prague Castle. Space solution with deeply based platform limited negative intervention to the valuable organism of the city on several temporary constructions. A surface entrance pavilion and atrium with a garden are a substitute for them. Their surroundings cultivated and brought new quality by enriching the historical heritage (Fig. 11).

Realization of the two stations *Můstek* with changes to line A and B was one of the most demanding in the network of Prague subway (Fig. 12). Station on line A was finished in 1978 and its part is also the underpass to Můstek. Nowadays it is a certain anachronism, but was adequate to the requirements of the times, during

Figure 9. The central railway station with subway station under the hall in the park.

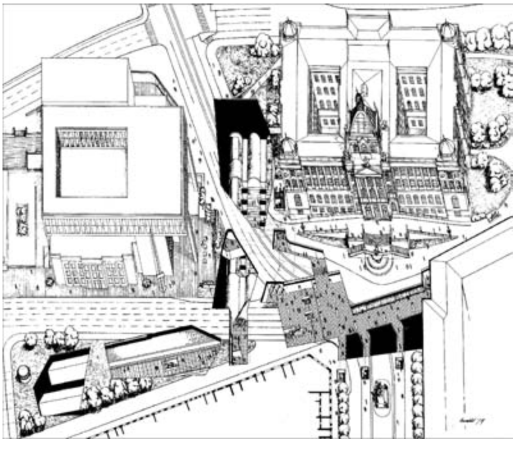

Figure 10. Axonometry of the change point by National Museum.

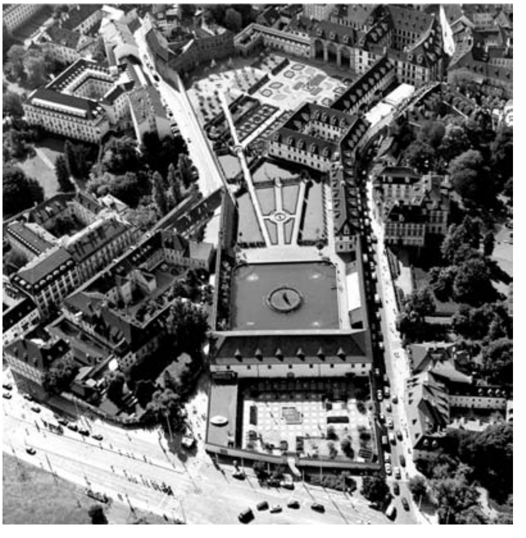

Figure 11. Malostranská station is a part of historical construction.

Figure 12. The diagram of the change point arrangement on Můstek station.

which it was constructed. A huge underground hall and the exit with the underpass to Jungmannovo square was also constructed together with finishing the station on B line in 1985. Shortly before its opening was decided that tram transport here is going to be cancelled and that car traffic is going to be significantly limited and that there is going to be a pedestrian zone. Both underpass areas are used as commercial zones (Fig. 13).

The construction of *Náměstí Republiky* station with deeply based platforms and two escalator tunnels was in the area concentrated on the western hall with the underpass under own square and to eastern hall with underpass to the Masarykovo railway station Free areas were used for their construction without demanding demolitions. Modifications of the parter on the square and foreground of the station were executed together with the construction.

Releasing once unsightly street V Celnici originated conditions for construction of new important hotels and multifunctional buildings in their surrounding, which created new urban cultural area. There is currently a construction of a big multifunctional complex Palladium in the area of former barracks and underpass will become the entrance area to the commercial area (Fig. 14).

Figure 13. Pedestrian zone under the underpass on Můstek station from 1989.

Figure 14. The area over the western hall of Náměstí Republiky station.

6 PRAGUE SUBWAY IN THE INNER CITY

Line C goes from the historical city centre to Pankrác terrace in the tunnel of the Nuselský bridge. On its southern foreground originated unique station Vyšehrad. The bridge was constructed together with the subway construction in 70's of the last century and became a connection of new city areas to the Pankrác plain with the central area. Thanks to this station once deserted place started to change its surroundings to new spacious city dominant with huge composition of Congress centre, hotels, office buildings and houses (Fig. 15).

A new centre grew by the station *Budějovická*. Projects on its construction originated simultaneously with subway projects and became mutually coordinated. The station was originally located on the free area, but a department store and sale gallery were established already during the subway construction. They were able to maintain fixed urbanistic conception for the creation of the centre of Budějovické square up to now and the square with the surrounding sets of offices, commercial buildings and houses is almost complete. This example of usefulness of good planning and coordination deserves to be followed (Fig. 16).

From many localities, where the subway initiated the reconstruction and development of the area, can be presented on the first place the new urban centre Smíchov and surroundings of *Anděl* station. There was still in eighties of the last century a big and old factory in the middle of Smíchov. Unattractive, mixed housing site remained in its surroundings. They managed to substitute in 1985 part of this housing site by surface pavilion of the northern hall together with realization of the subway station. The base of the new centre Smíchov became the Palác Zlatý Anděl, which was creating the new square along the Nádražní Street and originated

Figure 15. The bridgehead of Nuselský bridge with Vyšehrad station.

Figure 16. The Budějovické square with the subway station in the underground.

Figure 17. Centrum Smíchov over the subway station Anděl.

over the reconstructed northern hall (Fig. 17). The Southern hall of the station is placed in the area, around which is being prepared a new construction of urban sets on the currently problematically used areas. In the surroundings of Zlatý anděl gradually originated the centre of commerce, culture, accommodation, alimentation, housing, offices, garages and clubs even in more distant surroundings. Smíchov was really transformed into a modern city and the subway plays a major role.

Figure 18. A commerce centre grew over the subway station Chodov.

Figure 19. A new centre is growing by the hall of Roztyly station.

7 PRAGUE SUBWAY IN THE OUTER CITY

There were two stations on the II. part of C line – realized in almost free area. Both these stations evoked in the end of nineties a huge construction works in the surroundings of business, services, culture, sports and accommodation centres, administration and housing. A huge international technological park and commercial centre Centrum Chodov originated by the Chodov station, which is supposed to be the biggest in the Czech Republic (Fig. 18).

Station *Roztyly*, which was formerly called as the station for rabbits from surrounding forests, is nowadays important terminal for suburban and long-distance bus transport lines, a T-mobile centre was also created here and a big department store. A construction of the centre of a big travel agency, hotel and other buildings is prepared (Fig. 19).

Stations *Stodůlky*, *Luka*, *Lužiny*, *Hůrka* and *Nové Butovice* originated as natural local centres of housing sets on the South-western part of Prague. The preparation of the construction of this outer city began in the area, when it was already decided about the realization of subway in Prague. Authors could therefore create a conception of real *high-speed railway city*, which

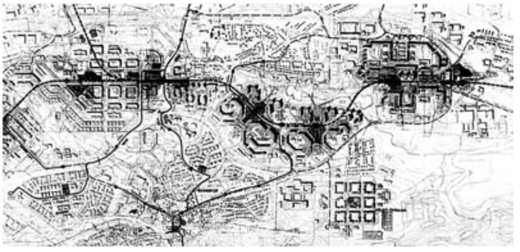

Figure 20. The plan of South-western City with a subway station.

does not have many parallels in the world (Fig. 20). They were successful to coordinate projects of the new residence area with projects for the preparation of subway construction so that a symbiosis of both buildings was created here. The construction of *South-western City* was quicker than the realization of the subway, but areas and corridors in determined localities were prepared for further construction of subway stations and track tunnels. Around stations in all localities were created natural centres for the life of inhabitants of housing complexes. The whole territory of South-western city is thanks to its transport availability in the interest of different, often multinational developers, so that 13. Prague district is the area, where many building of all kinds, importance and size, are constructed. A valuable and attractive city part was created from originally stark block of flats. Subway helped this development to a great extent (Fig. 21).

Rajská zahrada station and its original architectonic solution is inspiration for future. It is not exceptional in terms of traffic, but it is a modern investment to the values, according to which Prague is regarded as a beautiful city (Fig. 22).

Figure 21. South-western City from the East in 2003.

Figure 22. Station Rajská zahrada is an innovation in solution of stations.

Subway in an old town, such as Prague, represents a phenomenon, which helps the city in creation of the environment for its inhabitants, in the development of city's equipment, in using the historical centre and in its protection and preservation and broadening cultural heritage.

Source: Kyllar E., 2004. *Praha a metro*. For Inženýring dopravních staveb, a. s., published Gallery, Prague

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Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina, Romancov & Zlámal (eds)
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Railway line Prague centre – Ruzyně airport

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ABSTRACT: Prague, the capital of the Czech Republic, still does not have sufficient railway transport capacity serving the city centre and Ruzyně, the international airport in Prague. This should be changed by some 20 km of a new railway line which considerably uses the existing single-track line from Prague to the city of Kladno. In addition to the quick and reliable rail connection between the airport and the capital centre the modernised line will be accompanied with development of architectural and urban centres and important interchanges in the territory. The project comprises several tunnels which cover some 10% of the total length of the line.

1 BASIC IMPORTANCE OF THE RAIL CONNECTION

At present there is not quick, comfortable and especially reliable connection of sufficient capacity between the capital centre and Ruzyně, the international airport in Prague, provided by public transport. The existing bus transport is insufficient and with respect to the expected rise in the air transport volume (at present it is some 11 million passengers a year) it will be inefficient in the future.

From the point of view of a systemic nature it is definitely more suitable for the traffic channels of the airport if the railway transport is used as it removes the disadvantages of the bus transport with its limited capacity and poor travelling standard in general. Major airports in Europe and other countries all over the world are served by railway transport.

It is an advantage that it is possible to use the existing Prague – Kladno railway line, running through stations in Praha Dejvice and Praha Ruzyně. By modernising this line, its current substandard condition will be improved, incl. negative effects elimination such as noise, vibrations and the barrier effect (tunnels and elevated sections, grade-separated intersections). Despite the widening of the line (especially for the reason of doubling of the track) no acquisition of land of greater extent will take place. The new development of the line from Ruzyně station to the end station of Ruzyně Airport runs mostly in the existing traffic corridor (along the Prague ring road).

With respect to the prospective volume of transport when it is possible to expect that as at 2015 some 20,000 passengers will be transported in 24 hours in one direction (METROPROJEKT Praha a.s. 2002–2006), the railway transport is the optimum system as regards capacity.

At the same time the railway system provides a higher travelling standard, preferring seating room, luggage hold, allowing seat booking, etc. – in general it gets closer to the air transport standard. In addition to this the connection between the airport and the capital centre will be quick, regular and reliable.

The modernised railway line will also improve the standard of traffic channels of the biggest city of Central Bohemia and its region. This will be a through train connection without any need to change and therefore it will be competitive in comparison with individual car transport.

2 COMPLEX TRANSPORT RELATIONS, IMPACTS ON THE TERRITORY

The railway transport connection to Praha Ruzyně Airport creates conditions for its natural integration into the PID system and is fully in harmony with the policy of development of the traffic channels of the capital of Prague. The railway connection to the airport is of a radial and central character, the railway line ends right in the city centre, its historical core, Praha Masaryk Station. A possible connection of the airport to the long-distance transport can be dealt with in the future through further development of railway lines in the area of the airport.

The route runs through a highly urbanised area providing good conditions for being directly served and conditions for establishing relations with transports of other types. Nine railway stations and stops will be major traffic junctions and the most important

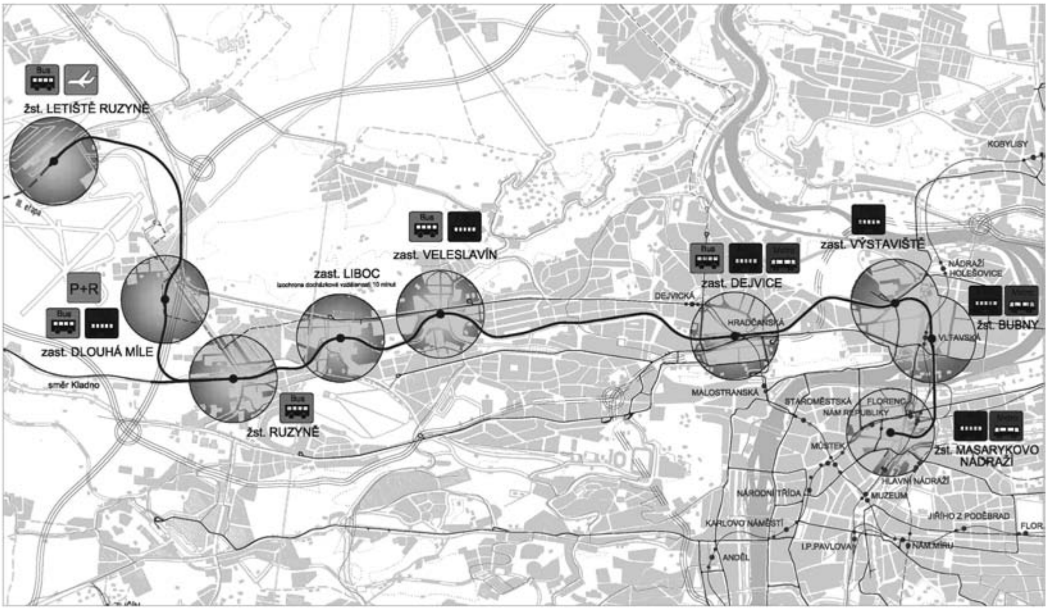

Figure 1. General layout of railway line.

ones – Masaryk Station, Bubny – Vltavská, Dejvice – Hradčanská and Dlouhá Míle – will also be natural architectural and urban centres of the particular locality. This transport connection will also provide an attractive change of all routes of Prague Metro (route A – Hradčanská station, route B – Náměstí Republiky station, route C – Vltavská station). Modernisation of the line will also eliminate all level crossings with the existing road network and thus improve the provision of transport services in the area in general.

3 PARTICULAR STATIONS AND STOPS

The starting point of the route, Praha Masaryk railway station, is situated in the very capital centre, allowing the possibility of changing to the metro route B and tram lines. The station is directly linked to the adjoining area which is important especially as regards accessibility of services and job opportunities (shopping and office centres, major accommodation capacities, etc.). The development of the line and its terminus in the premises of Masaryk station will also considerably contribute to the improvement of the devastated northern part of the station where extensive conversions are planned in connection with the introduction of the line.

The newly planned Bubny – Vltavská station is part of the development area of Holešovice – Zátory and is of an important potential as an interchange point and a destination of trips in relation to the newly planned urbanisation of this area. It is expected that the adjoining area will meet residential and servicing purposes. A new northern hall of the route C Vltavská metro station will be built there and a tram stop will be near.

The new overhead Výstaviště stop will be of its greatest importance especially as regards access to the premises of Výstaviště (Exhibition Ground) and the Stromovka Park (especially as regards services, leisure time activities, culture and sports). At the same time the particular area should be improved from architectural and urban point of view. There will be a direct connection to the tram transportation.

Dejvice – Hradčanská underground stop will provide a short connection to Hradčanská station of the metro route A. At the same time a construction of the metro station western hall is planned. It will also be possible to change to trams and busses. Next future intensive urban development of the area between Hradčanská metro station, the existing Praha Dejvice railway station and Prašný most (Powder Bridge). The station will thus be important as regards availability of services and residential zones. The fact that the station will be underground will improve availability of Prague 6 to pedestrians and in general it will allow its urbanisation.

The partly underground Veleslavín stop moved to the north will be close to Evropská avenue and will make changing to trams or busses possible. Close to the station should be built a new multipurpose building. The station will thus serve also the adjoining area.

The new Liboc stop will both serve the adjoining area (especially residential areas) and allow indirect pedestrian connections to Evropská avenue and the holiday areas of Divoká Šárka and Džbán.

Ruzyně station, moved in the western direction, will be in a space contact with the new grade-separated intersection with Drnovská street, where a bus stop could be established. The station could be important also as regards accessibility of adjoining storage premises. It also creates conditions for urbanisation of the surroundings and allows its direct servicing. Behind the station the line branches off, one branch runs to the airport and one to Kladno, passengers coming from Kladno can change for the airport (until the development of the direct line from Kladno to the airport).

The new Dlouhá Míle stop is an important terminal in the north-western part of the capital where all bus services coming from this direction should end. At the terminal it will be possible to change to the public busses as well as to the trams whose stop will be built in the future. Part of the part of the junction will also be a capacity car part of the P + R system for the purposes of the individual car transport, close to the ring road. Dlouhá Míle terminal is thus the basic junction in this part of the capital owing to which it is possible to reduce the non-railway transport (BUS, individual car transport) in the territory of Prague 6.

The new Ruzyně Airport end station, in the premises of the international airport of Prague, is directly connected to the existing and new terminal and other structures within the airport halls. At the same time it is possible to change to busses here and it is also the required element of integration of the air transport with the PID capital transport system of a corresponding functional and quality standard.

4 BASIC DATA

- Length: 20.3 km, with 5.6 km newly built section Praha Ruzyně railway station – Praha Ruzyně Airport railway station.
- Total number of stations (stops) 9; it is completely a double-line railway.
- Design speed: 80 km/hour.
- Min. curve radius R = 325 m, max. gradient 25‰.
- Traffic completely as electric traction (direct-current, 3000 V).
- Regular shuttle service of the airport and Kladno trains, the interval corresponding to the demand and the chosen transport standard. The peak interval for airport: 10min., for Kladno: 20min.
- Journey time from Praha Ruzyně airport to Masaryk station: 26.5min (stopping train).
- Journey time from Kladno – Masaryk station 40min (stopping train).

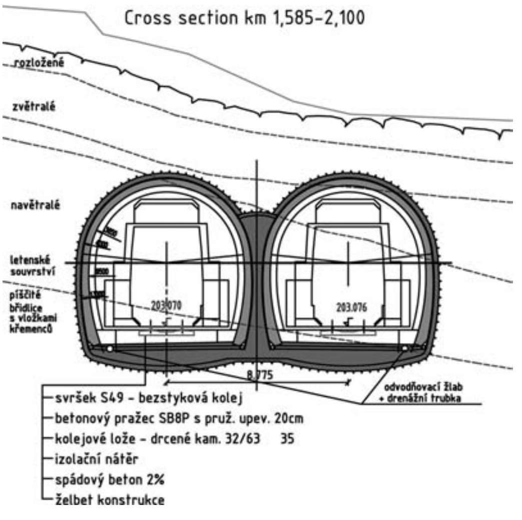

Figure 2. Cross section km 1,585–2,100.

5 UNDERGROUND STRUCTURES ON THE ROUTE

It is not possible to build a structure of this extent within the capital without underground sections of which there will be a great number.

5.1 *Tunnels in the area of Stromovka Park*

If proceeding in the direction from the capital centre, the first tunnels are designed in the Stromovka Park in length of 800 m. They are designed here to minimise negative effects of the new route on the environment. Stromovka is listed as a cultural monument and it is a nature park. The section of tunnels starts with a portal right outside the vaulted overbridge adjoining Kamenická street. The portal is situated next to the overbridge so that the overbridge will be preserved. Tunnels of the double-track line pass partly under the park and partly under the development adjoining the park and are designed on principle as single-track tunnels with a common supporting pillar. This solution with a minimum axial distance of rails allows minimisation of the impact on the outside environment, whether it covers the width of the influenced zone as regards the development area and valuable park area or also overlying rock drifts and surface drops.

Another section is the subway under the existing bridge in Korunovační street. In this place one tunnel tube runs under the existing line and under the right pier of the bridge. If full operation of the bridge is to be maintained, it will be necessary to underpin both its piers of the bridge and secure its right support for the driven subway. Both the tunnel tubes will be gradually

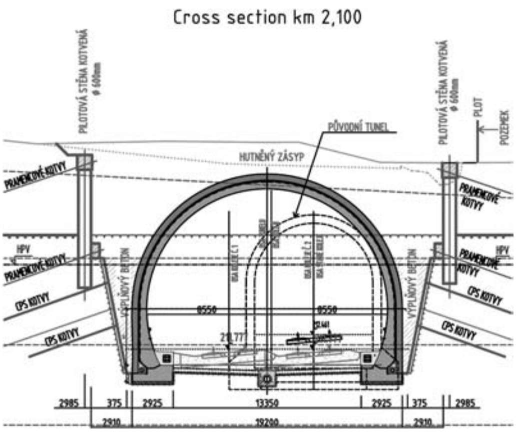

Figure 3. Cross Section km 2,100 – another variant.

inserted, along the bridge subway, under the railway cut so that 1.5 to 2.0 m of rock remain above the top section. In this section the driving of the left tunnel tube will be combined with the development of the right tunnel tube under the protection of the concrete ceiling which will be supported, in the area of the common intermediate pillar, with a number of root piles, which will become part of the common intermediate pillar of the primary lining of both tunnel tubes.

Tunnels will be driven in layers of shale of Letná. The tunnelling and the primary supporting of the tunnel tubes or the intermediate adit is to be performed in accordance with the principles of the New Austrian Tunnelling Method.

Another discussed variant of this passage through Stromovka is a mere doubling of the track of the existing line and its partial covering. This solution requires the necessity of reconstructing the existing single-track Dejvice tunnel, approx. 100 m long, which is a listed structure (the oldest railway tunnel in the Czech Republic). This tunnel had been built in a cut and then covered. The new tunnel of the required arched shape would have to be built also cut-and-cover with pile walls shoring of the building pit. On the basis of experience of similar structures the vaulted, arch structure of an excavated tunnel is built by means of formwork formed from inside with a formwork vehicle and from outside with a system formwork or with auxiliary primary lining.

This variant also requires a complete reconstruction of the bridge in Korunovační street with necessary stoppage of traffic on the bridge.

5.2 *Cut-and-cover section in Letná*

This section is adjoined by a 640 m-long section of a cut-and-cover double-track tunnel in Letná, situated right under the surface. It runs as far as the underground Praha Dejvice-Hradčanská station. It is a cast-in-situ reinforced concrete one-nave frame in an open cut. This solution allows a grade-separated intersection with several cross streets outside the station. The tunnel structure is close to the structure of the exit ramp of the tunnels of the City Ring Road (Ramp 1 – Letná City Ring Road) and the progress of both the building activities will have to be coordinated with respect to the course of time of both the structures.

5.3 *Praha Dejvice – Hradčanská station*

The new Praha Dejvice – Hradčanská underground station will be situated between Milady Horákové avenue, Dejvická street, the existing ČD Praha Dejvice station and Bubenečská street in direct relation to the existing hall of the metro station. The space around Hradčanská metro station and the new station is one of the most complex places on the route. The development of the new underground cut-and-cover station is in close contact with the planned development of road tunnels of the City Ring Road.

It will be connected to the metro station on the level of the existing subway of the metro station from which it will be possible to get into the middle of the railway platform. From the north-western end of the platform of the new station it will be possible to change to metro or leave for Dejvice through the newly planned western metro vestibule. The perimeter walls of the station are diaphragm walls, which will also shore the foundation pit. They are strutted with the foundation plate of the station, on the upper level partly with concrete floor structure, partly anchored with permanent anchors and partly strutted within the final glazed steel structure of the ceiling above the platform.

5.4 *Cut-and-cover tunnels behind Praha Ruzyně station*

In the section between Praha Ruzyně station and Praha Ruzyně Airport end station there are planned several sections of cut-and-cover tunnels where it is not possible to run the route on the surface for the reason of crossing the road beds and protecting airport systems (impact of electric traction on the radio beacon, protection of the route under the planned runway) on this edge of the city. There are altogether five sections of the total length of 1250 m.

Tunnels are designed as cast-in-situ reinforced concrete frame which is not closed in the lower part, longitudinal strip foundations are interconnected transversely only with ribs of max. distance of 8 m. Inside dimensions were determined on the basis of the maximum moving dimensions and requirements for drainage of roadbed. They will be built in an open pit either sloped or shored by Stent Walls.

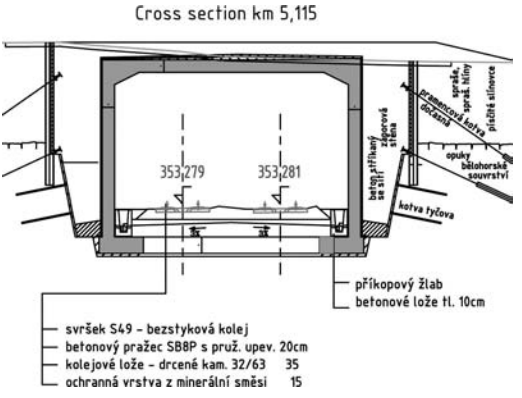

Figure 4. Cross Section km 5,115.

5.5 *Praha Ruzyně Airport station*

This end station with an intermediate platform is situated in the area between air terminals along the elevated roads. The station is the underground one, the bearing structure is a closed cast-in-situ reinforced concrete frame. The station area will be, in addition to the direct exit leading to the surface, also interconnected with the airport buildings with two underground connecting corridors – the eastern one connecting the station with the original terminal and the western one connecting the station with Sever 2, the recently completed new terminal. For the purposes of this corridor, in connection with the construction of the new terminal there was built an underground advanced structure which will allow completion of the corridor without any impact on the surface.

The station will be built in a cut-and-cover system. It will be necessary to anchor gradually the piers of the parallel elevated road. The elevated road was founded so that it would be possible to excavate the pit for the station structure.

6 CONCLUSION

At present the project documentation is being discussed at the stage for planning proceedings and number of additional studies are being completed.

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The Trans Hudson Express (THE) Tunnel

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ABSTRACT: Meeting New York Metropolitan area mobility needs is critical to keep pace with growing demand, to achieve the forecasted economic growth, and to maintain midtown Manhattan as a centre of regional, national and global importance. The possibility of insufficient access to midtown Manhattan was recognized as a significant problem, leading to this project major goal of doubling trans-Hudson rail capacity. With a budget of over six billions dollars (2005) and a completion plan by 2016, the Trans Hudson Express (THE) Tunnel project will fulfil these needs. The project faces many technical, environmental, and regulatory challenges including tunnelling under the Hudson River in soft silts and with limited cover; tunnelling through mixed ground in the New Jersey shore; crossing under existing historic bulkhead; tunnelling under high rise buildings in one of the most congested region; constructing a six-track cavern station under 34th Street, one of Manhattan's busiest cross-town thoroughfare; underpinning and supporting existing facilities; maintaining NJ Transit, Amtrak, Long Island Rail Road and New York City Transit operations during construction; and developing the design concepts to meet environmental requirements and regulatory constraints. This paper presents the planning process of the project, the environmental and engineering challenges, and measures taken to address them.

1 GENERAL

The New York/New Jersey metropolitan area is one of the most densely developed areas in the United States. Within the trans-Hudson Midtown corridor, almost 300,000 people commute into Manhattan each weekday on various modes from counties west of the Hudson River. This demand has grown to exceed available capacity on the trans-Hudson transportation network which includes roadways, buses, ferries, PATH trains, and commuter rail to Penn Station New York (PSNY), the existing trans-Hudson rail tunnels, and within PSNY itself. The consequence is that NJ TRANSIT's (NJT) options to provide additional rail service into midtown Manhattan during peak hours have been exhausted without major new infrastructure. Compounding the situation is that demand for additional public transportation solutions are certain to increase, as population and employment across the region grow. Recent projections for commercial development in Manhattan estimate more than 200,000 additional office-sector jobs will be created over the next two decades. Furthermore, maintaining a safe and secure rail system is the highest priority of rail operators. The post 9/11 attack security environment demands rail redundancy across the trans-Hudson Midtown corridor, where one incident can quickly impact hundreds of thousands of commuters, to be critical.

Presently, NJT operates 23 trains per hour during the morning and evening peak periods to and from Penn Station carrying about 42,000 passengers each way during the peak period and about 140,000 passengers per day each way. By 2025, ridership is expected to increase by about 70% to more than 200,000 passengers per day. The Trans Hudson Express (THE) Tunnel project, formerly known as the Access to the Region's Core (ARC) project is vital to accommodate the increased ridership. THE Tunnel will support the New York/New Jersey metropolitan area's future economic well-being by improving trans-Hudson mobility, maintaining a safe and secure transit system, maintaining and enhancing the economic viability of the region, improving the quality of life for transit passengers, and preserving and protecting the environment.

2 PROJECT DESCRIPTION

The project consists of providing additional tracks in New Jersey along the North East Corridor (NEC), new tunnels across the Palisades and the Hudson River, new tunnels in Manhattan, and a new cavern station under 34th Street. See Figure 1. In New Jersey the project consist of the construction of loop tracks connecting the Main/Bergen/Pascack Valley lines to the NEC near the Secaucus Junction Station, to provide a one-seat ride to and from midtown Manhattan from northern New Jersey, and from Orange and Rockland Counties in New York; a new rail yard in Kearny to accommodate 45 to 50 train sets; and improvements along the NEC to increase its capacity. New tunnels under the Palisades in New Jersey and the Hudson River will connect to a new 34th Street Station in New York and the existing PSNY. The new station will be connected to the existing PSNY, and the Sixth, Seventh, and Eighth avenues, and Broadway Subway lines. With a budget of $6.1 billion (2005) and anticipated completion date of 2016 the project will provide infrastructures, facilities, and rolling stocks to accommodate a total of 48 trains per hour to New York carrying more than 86,000 riders each way during the morning peak period. In addition, NJT is implementing, independently from this project, improvements in PSNY to accommodate the increase in capacity and ridership. These improvements include platform extensions, vertical circulation elements, track modifications and additional storage capacity.

2.1 *Project alignment*

The project extends from just east of the Hackensack River on the NEC to West 34th Street and Sixth Avenue in New York City and includes a new rail yard in Kearny. The alignment includes new loop tracks (Secaucus Loop) from the outer tracks of the existing Main Line on the lower level that connect with the upper level NEC tracks west of Secaucus Junction Station. The new loop tracks will utilize the former Boonton Line right-of-way, and allow Main-Bergen/Pascack Valley Line trains to continue to PSNY or the proposed 34th Street Station through either the existing North River tunnels or the new tunnels. This arrangement will create a direct, one-seat ride for passengers from Northern New Jersey to New York City.

East of the Hackensack River, the alignment includes a new bypass track that would split off to the northern side of the NEC, creating four tracks along the NEC from the Hackensack River to the new tunnel portals. This would allow express service along the NEC in either direction. Two tracks would connect to the existing North River tunnels, and two tracks would connect to the new proposed tunnels, descending and turning southward under the Palisades through Union City and Hoboken. Interlocking configurations along the NEC are developed to provide full flexibility between the existing tunnels and the new tunnels

Figure 1. Alignment in New Jersey.

Figure 2. Alignment in New York.

to facilitate emergency operations or periodic closures of the tunnels for maintenance. See figure 1.

Near the Hudson River in Hoboken, the new tunnels will turn eastward to cross the river. At the eastern shore of the river in New York, the alignment will begin to ascend in grade, and turn in a northeast direction, passing under the Hudson River bulkhead. Connections to PSNY will split from the main tracks after passing the bulkhead. The PSNY connector tracks will continue to ascend, before turning east to tie into existing PSNY tracks. Beyond the point where the PSNY connector tracks will split off, the main tracks leading to a new 34th Street Station will descend, and split to four trackways, arranged two-over-two. At West 34th Street, the alignment will turn eastward to match the street grid. Each tunnel level will split into four platform tracks serving two island platforms in a two-cavern station. See figure 2.

3 PROJECT GEOLOGY

The New Jersey portion of the project area is located within the Piedmont physiographic province broad

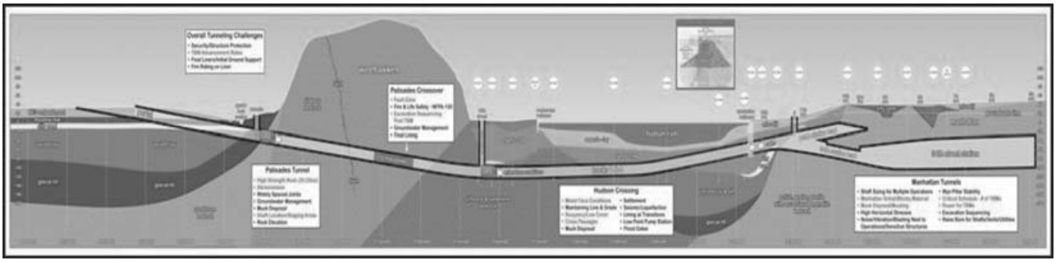

Figure 3. Geological profile.

lowland interrupted by long, northeast-trending ridges and uplands. The most prominent physiographic feature in the eastern part of the province is the Palisades, a striking north-south topographic ridge near the Hudson River that rises above the surrounding lowlands of the Meadowlands. Most of the project area in New Jersey is underlain by rocks of the Newark Basin, a northeast-trending late Triassic – early Jurassic rift basin filled with a thick sequence of sedimentary rocks and intrusive and extrusive igneous rocks. The topography of the bedrock surface shows two narrow, deep, glacially scoured troughs, one on either side of the NJ Meadowlands. Metamorphic rocks in the project area occur only along the Hudson River waterfront in Hoboken and Jersey City. The Hartland Formation consists of gray, interbedded schist, granulite, and amphibolite.

Thickness of surficial materials in the project area in NJ varies from one meter to greater than 75 m (250 feet) at a glacially eroded bedrock trough in the vicinity of the New Jersey Turnpike east of Kearny. Surficial materials consist of deposits of glacial, eolian, alluvial, and marsh/estuarine origin. The Rahway till is a surficial unit directly overlying bedrock. It is a non-stratified, compact deposit with pebbles, cobbles, and boulders in a reddish-brown matrix of sand, silt, and clay. Its thickness is generally less than 10 m (30 ft). Overlying the till are glacial deposits along the west flank of the Palisades ridge, at scattered locations near the Hudson River, and near the Passaic River. The unit includes both deltaic deposits of sand, sand and gravel, and silty sand and lake-bottom deposits of fine sand, silt, and clay. Thickness ranges from about 8 m (25 feet) to over 30 m (100 feet) in the Meadowlands east of Kearny. Passaic terrace deposits, consisting of moderately sorted sand and gravel, are present along the Passaic River in the vicinity of Newark and Harrison. Light brown eolian deposits of very fine to medium sand occur locally near Laurel Hill and just west of Penhorn Creek. A large percentage of soils in the project area have been altered by excavation or filling. Earth and manmade materials that have been placed as fill include gravel, sand, silt, clay, trash, cinders, ash, and construction debris. In the cut and cover section in New Jersey the soil stratigraphy from the ground surface consists of 2.5 to 3 m (8 to 10 ft) of marsh deposits, underlain by about 1.5 m (5 ft) of alluvial sand, underlain by about 6 to 12 m (20 to 40 ft) of hard blue clay deposits of glacial Lake Hackensack, which are in turn underlain by 15 to 24 m (50 to 80 ft) of hard red to brown clay deposits of glacial Lake Bayonne. In the centre of the trough, bedrock is overlain by a glacial deposit of till or hard sand and clay about 15 m (50 ft) thick.

The Hudson River portion of the project area is located between the Piedmont physiographic province on the west and the Manhattan Prong of the New England Upland physiographic province on the east. The topography of bedrock surface underlying the Hudson River shows a narrow, deep, glacially scoured trough that extends to more than 100 m (300 ft) below sea level. Depth to rock is about 40 m (125 ft) below mean low water at the New York bulkhead line and about 55 m (180 ft) below mean low water at the New Jersey bulkhead line in Hoboken. The maximum total thickness of surficial materials overlying bedrock of the Hudson River in the project area is about 100 m (300 ft), with a complex stratigraphy of glacial, fluvial, lacustrine, and estuarine deposits. It is anticipated that the soil materials through which the Hudson River tunnel would be constructed would be primarily estuarine deposits of organic clayey silt and fine sand, with sand or glacial till possible near the New Jersey waterfront and sand or organic clay near the New York waterfront. Gas pockets due to natural methane have been reported in the estuarine silts, mostly at shallow depths. It is also anticipated that hidden offshore obstructions within soil materials, such as boulders, piles, wrecks, or miscellaneous fill exist on both sides of the Hudson River.

The New York portion of the project area is located within the Manhattan Prong of the New England Upland physiographic province. Within the Hartland Formation, granitic intrusions are present near the Hudson River. A large granitic sill has been mapped between Ninth and Twelfth Avenues, and between West 34th Street and West 40th Street. Serpentinite associated with the Hartland Formation has been reported between Tenth and Eleventh Avenues, at the northern part of the project area, and at scattered locations as far south as West 26th Street. Thickness of surficial materials is variable and they generally have been altered by excavation, filling, or paving for various developments. The entire length of the Hudson River waterfront in the project area is reclaimed beyond the original mid-19th Century shoreline. Filled for urban development, these areas are typically former bays or tidal marshes with organic deposits beneath the fill. Most of the Manhattan tunnels east of Eleventh Avenue, are expected to be excavated primarily in schist of the Hartland Formation, including some granite or pegmatite and possibly serpentinite. Rock along the proposed alignment in this section is likely to be moderately weathered and of fair to good quality within the first 1.5 m (5 ft). Below this zone, weathering is likely to be slight, with good to excellent rock quality. The schist is typically of intermediate strength, with unconfined compressive strength generally ranging from about 28 to 100 MPa (4,000 to 15,000 psi). It may contain significant percentages of abrasive accessory minerals such as garnet or sillimanite, as well as thin veins of quartz. The granite or pegmatite is likely to be of high strength, with unconfined compressive strength from about 100 MPa (15,000 psi) to greater than 170 MPa (25,000 psi) and it is likely to contain a very high percentage of abrasive minerals, mostly quartz. Groundwater levels are within the overburden about 3 m (10 ft) of the ground surface and generally parallel to the bedrock surface. Rock mass permeability is relatively low but with higher permeability and greater water inflow along weathered joints or fault zones.

4 34TH STREET STATION CAVERN

The proposed station concept incorporates a potential four-over-four track configuration in two caverns with two tracks and an island platform over two tracks and an island platform in each. Initially six tracks with three island platforms will be provided. A mid-level mezzanine would be provided in each cavern to provide for circulation and pedestrian distribution. The two caverns will be connected to each other with cross passages and will be connected to PSNY through the 33rd Street LIRR concourse. See figure 4. Street entrances along 34th Street between Eighth and Sixth Avenues will be provided.

The conceptual design incorporates belowground connections to the Broadway, Sixth, Seventh, and Eighth avenues subway lines. The station caverns will be approximately 20 m (60 ft) wide by 23 m (70 ft) high and would be placed about 35 m (110 ft) below street surface at the mid-level mezzanine elevation. The southerly cavern will be located under 34th Street, and

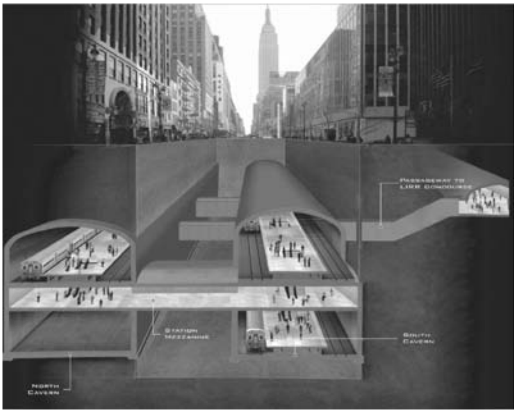

Figure 4. 34th Street station concept.

the northerly cavern will be located under properties along the northern side of 34th Street. The caverns crown will be located approximately 17 m (50 ft) below the top of rock to provide suitable rock cover.

5 TUNNELING CHALLENGES

Building the Trans Hudson Express project will involve construction of a number of tunnels using a variety of construction methods. Selection of an appropriate tunnelling method for each section of the alignment will have a major impact on the successful completion of the project. The project faces many technical challenges:

- Selecting an alignment to meet the transportation needs and minimize environmental issues, while meeting tunnelling method technical challenges.
- Developing a versatile, yet economical and constructible tunnel cross section capable of meeting present and future equipment, and accommodating the required utilities and ventilation.
- Developing track vertical alignments that meet the required design grade to provide connectivity to the existing PSNY and to provide sufficient cover over the tunnel under the Hudson River.
- Utilizing suitable construction techniques in soft silts of the Hudson River with a limited cover.
- Constructing a new cavern station in New York City with access to the existing PSNY, and the existing transportation systems in New York, while minimizing impact on the surface, businesses, and the public.
- Identifying locations for construction staging especially in Manhattan and meeting the needs of other projects in the area.
- Identifying provision for emergency egress and ventilation with minimal impact on land use.

- Minimizing impact on the Hudson River, the Hudson River Park, future developments and existing and future facilities along the alignment.
- Tunnelling in mixed face conditions under the Hudson River.

5.1 *Tunnelling methods*

The construction of the alignment in New Jersey along the NEC would mostly be constructed using the existing or an enlarged embankment, viaduct structures, retained fill, and cut-and-cover tunnels. As the alignment diverges from the NEC and dives under the embankment the construction would be done either by cut and cover, tunnel jacking, or by using sequential excavation or NATM. Cut and cover method will be used to construct other underground segments of the New Jersey alignment to the tunnel shaft west of the Palisades. The cut and cover construction will use slurry walls for support of excavation and water cut off. Staging, rerouting, and decking will be provided to maintain traffic across the excavation. The Palisades tunnels will be constructed using rock TBM starting in a shaft located near Route 1&9. The shaft will be used for access, material delivery, and mucking. Upon completion of the construction it will be converted to a ventilation shaft where a ventilation facility will be located. The tunnels are located about 120 m (350 ft) below surface in the Palisades diabase, with argillite and hornfels of the Lockatong formation for short lengths on either side of the diabase. A cross-over will be constructed between the two tunnels using controlled drill and blast method utilizing the previously excavated tunnels. Crossing the Hudson would be done by a pressurized face soft ground TBM using either Earth Pressure Balance (EPB) TBM or slurry TBM. The TBM will be lowered in a shaft in North Hoboken and would mine the tunnels toward Manhattan under the Hudson River. The tunnels will be lined using one pass gasketed and bolted precast segmental liner. As the alignment approach Manhattan, the cover over the tunnels is limited and special measures would be needed as discussed later. The TBM will be retrieved via a cofferdam constructed in the river at the edge of the bulkhead. Cross passages would be required under the Hudson River that will be constructed using NATM with localized ground freezing. Crossing the river bulkhead, the Hudson River Park, and 12th Avenue in Manhattan would be done using either the cut and cover method with decking and staging to maintain vehicular and pedestrian traffic or alternatively soft ground NATM can be used to construct the crossing with jet grouting and/or ground freezing for ground improvement. The Hudson River TBM can then continue through the previously constructed NATM tunnels and retrieved on land.

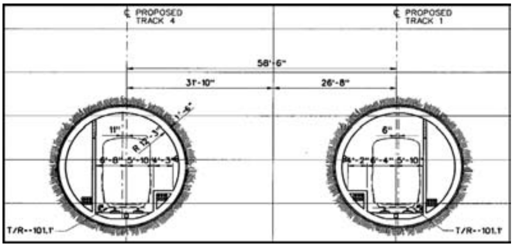

Figure 5. Tunnel cross section.

Tunnelling in Manhattan is challenging in term of access and staging. It is planned that most of the tunnelling in Manhattan would be done from a staging site at 11th Avenue. Two rock TBMs would be used to excavate four tunnels along the alignment through the station caverns. After the initial two runs, the TBMs will be retracted through the tunnels to the 10th Avenue interlocking caverns where they will be re-launched again to excavate the second pairs of tunnels. Subsequently the caverns will be excavated by sequential excavation using controlled drill and blast method from the mined tunnels to create the complex underground track configurations as the tunnels split to serve potential eight-track station. Access and mucking will be through the previously excavated tunnels. The initial liner in the rock tunnels will consist of rock bolts, lattice girders and shotcrete and the final liner will be cast in place concrete liner over a drained waterproofing system.

5.2 *Tunnel cross section*

The tunnel size was established to allow all available vehicles through the tunnel. Its size was based on the Amtrak Super-liner locomotive, with a 4.928 m (16′-2″) vehicle height. The inside tunnel diameter was established to be 7.47 m (24′-6″) to accommodate in addition to the largest equipment, the catenary power requirements, a suitable emergency walkway, and the required systems and utilities. See figure 5. This size tunnel allows the use of the available space for a ventilation plenum created within the bored tunnels to meet NFPA 130. The plenum would be equipped on the top side with motor operated dampers at certain intervals to be used to exhaust smoke in case of fire. In the event of an incident, the dampers in the immediate vicinity would be opened to extract the smoke and hot gases from the trainway. The tunnels will be spaced 17.8 m (58′-6″) on centre to allow sufficient pillar between them especially in the soft silts under the Hudson River.

5.3 *Limited cover over the Hudson Tunnel*

There are several challenges that constrained the design of the connection to existing PSNY. The engineering alignment underwent many alternative designs to address these challenges. The connection to PSNY was only possible south of the existing North River Tunnel due to the LIRR yard access tracks north of the tunnels and NJT's use of the southern half of PSNY. The operational analysis indicated that 2.0% is the maximum allowable effective grade for locomotive-hauled trains; however with such grade the cover over the tunnels under the Hudson River is inadequate. In an attempt to address this issue two possible solutions were evaluated: an increase in the grade to 3% for approximately 600 m (1800 ft) or artificially increasing the effective cover over tunnel by providing suitable ground improvement. A maximum grade of 3% may be permitted on the PSNY connection if equipment is limited to EMU train sets with multiple-powered coaches. However this will limit the operational flexibility of the railroad. On the other hand, intervention in the Hudson River would trigger environmental and permitting issues. For an effective grade of 2.5% the cover over the tunnel will be about 5 m (15 ft). In this case grouting a canopy over the tunnel from within the TBM can be provided to stabilize the ground and to protect the tunnel face against stability and buoyancy. The TBM would then advance under the protection of the grouted canopy without impact on the Hudson River.

5.4 *Crossing the Hudson River bulkhead*

The Hudson River Bulkhead is constructed of sheet pile walls with stone façade and timber relieving platform and supported by timber piles 18 to 30 m (50 to 90 ft) deep spaced about 1 m (3 ft) apart. The bulkhead is part of the Hudson River Park and is considered a historical structure. Therefore, it is important to preserve and protect it to the maximum extent possible. The piles will intercept the tunnel profile and are expected to be at variable spacing at the tunnel level. Three different methods have been conceptually evaluated for intercepting the Hudson River bulkhead and for constructing the tunnels in the Hudson River Park: continuation of the Hudson River Tunnel Boring Machine through the timber piles with special measures if needed, extraction of the piles, cut-and-cover construction, or excavation using the NATM method with ground improvement.

5.5 *Connection to existing Penn Station*

In order to connect to existing tracks in Penn Station, the track alignment in Manhattan at 12th Avenue diverge to four tracks. The inner tracks descend toward the new station at 34th Street while the outer tracks ascend to meet the existing tracks at the approaches to Penn Station. The track connections to PSNY pass under the Long Island Rail Road (LIRR) storage yard and the LIRR maintenance shop at 10th Avenue in close proximity to their foundations. These tunnels will be constructed using the cut and cover method or NATM. Special measures will be used to minimize the impact on the yard and the shop operations. The yard tracks and the shop will be underpinned prior to the excavation of the tunnels. Underpinning and pre-treatment of the ground will be done at night and on weekends to minimize impact. Conventional tunnelling will be done beneath the yard and the shop with extensive monitoring.

6 CONSTRUCTION COST AND SCHEDULE

At a budget of $6.1B (2005) construction is expected to begin in 2009 and to be completed with revenue operation by late 2016. The peak construction is expected to be in 2011. In order to establish a reliable construction schedule and cost estimate an initial construction packaging plan was established. The plan identified sixteen different contracts ranging in sizes and durations. The packaging plan took into consideration the type of construction, size of the contracts, and their durations and dependency on each other. The Manhattan segment with the TBM tunnels, the 34th Street Station caverns, and subsequently the railroad system work and the station finishes are on the project critical path.

7 CONCLUSION

The Trans Hudson Express (THE) Project is the largest transportation project ever undertaken by NJ TRANSIT. It will improve the overall New York metropolitan area transportation system, will stimulate economic growth, and will maintain midtown Manhattan as a centre of regional, national and global importance. The project has been planned using state-of-the-art design concepts, latest construction techniques and approaches, and with careful attention to environmental issues and concerns.

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CTA Block 37 Tunnel Connection in downtown Chicago

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ABSTRACT: Block 37 Station Tunnel Connection project will include construction of two short tunnels to connect the existing subway tunnels (Red and Blue lines) in the northern loop area of downtown Chicago through a new station. The Southeast Tunnel will connect the Red Line to the new station below the intersection of State and Washington Streets and the Northwest Tunnel will connect the Blue Line to the new station below the intersection of Dearborn and Randolph Streets. The connection tunnels are located in a very dense urban area of downtown Chicago and under several important historical buildings. Both tunnels are curved at the intersections and located at a depth of about 7.5 m below ground surface in saturated soft clay with an average undrained shear strength of approximately 25 kPa. The design calls for minimizing surface disruption and ground displacement while maintaining normal operation of the existing subways during the construction of the connections tunnels.

1 INTRODUCTION

To create an express train service linking O'Hare International and Midway Airports, the Chicago Transit Authority (CTA) decided to build two short tunnels to connect the Blue and Red transit lines in Chicago. The connection will be at Block 37, located between State and Dearborn Streets, and will link the two subways through a new station (Figure 1). The new Block 37 station structure will be constructed as part of the Block 37 development project using slurry wall construction

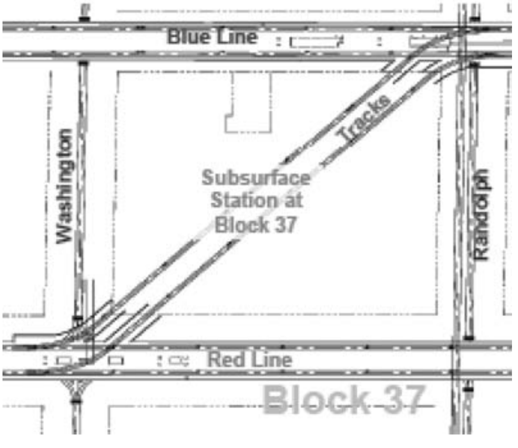


Figure 1. Plan view of project site.

and a top down technique. The new connection tunnels will be constructed using mining methods. Mining operations will originate from access shafts to be built within Block 37. The project will also include modifications to the continuous platforms at the existing State Street and Dearborn Street subway tunnels to accommodate the new connections. This project will be accomplished under a complex and tight schedule and must be completed with minimal disruptions to the non-stop subway service. The anticipated subsurface conditions and the stringent project settlement criteria create difficult conditions for the design and construction of the tunnels.

The soft deposits at the project site with their low shear strength can experience large deformations during tunneling. Application of a pre-support interlocking pipe arch method and fiberglass face reinforcement technique ahead of the advancing tunnel face are proposed to limit the ground deformation.

Continuous and swift excavation and support installation combined with an appropriate staging and support system is necessary to minimize short and long-term deformations. The tunnels will be constructed using fiber reinforced shotcrete and lattice girders as an initial support lining. The final lining will be installed using a composite system of steel fiber reinforced shotcrete and lattice girders.

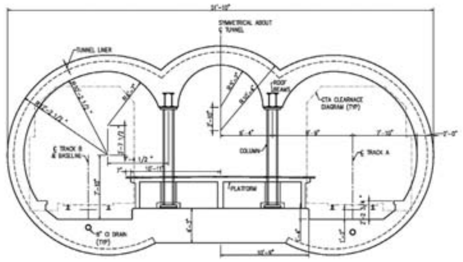
For the connection of the new short tunnels to the existing subway tunnels, a new larger span tunnel will be constructed using the interlocking pipe arch technique which will be parallel to the existing Tunnel. The new expanded tunnels will transfer the imposed loads to the column line of the existing tunnels. To facilitate the construction of the new expanded tunnels, two cross adits will be constructed perpendicular to the existing tunnels. These cross adits will originate from within Block 37 and extend below State and Dearborn Streets and above the existing Track B and Track C. This construction approach allows all tunneling work, including excavation, temporary and final lining work, to take place outside of the existing tunnel envelopes. Revenue operation can continue practically uninterrupted until the final breakthrough of the existing tunnels is required.

Structural modification and reinforcement of the existing tunnels must take place prior to the excavation of the new tunnels. These structural modifications and reinforcement consist of platform demolitions, erection of new tunnel roof structural support systems and installation of permanent tie down anchors. Removal of some of the existing columns and longitudinal beams will take place after construction of the new tunnels and breakthrough of the sidewall of the existing tunnels. Most of these construction activities will be performed using enclosures separating the work areas from the revenue operations.

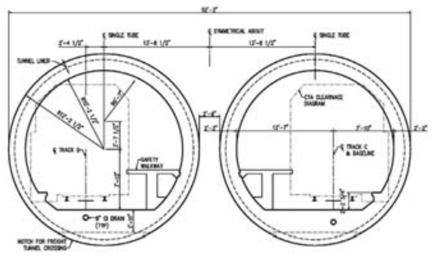
2 EXISTING SUBWAY TUNNELS

The initial subway system of Chicago included two routes, State Street Route and Dearborn Street Route. The tunnel portion consisted of 7.7 miles of double tube and station sections. All the tunnels were entirely constructed in Chicago clay. The bottom of the tunnels is located at a depth of about 15 m below the street surface.

For constructing the tunnels two different methods were used: the liner plate method and the shield method. In the liner plate method the pressure which acts on the roof of the tunnel is carried by steel ribs which transfer the load onto the footings. In order to use this method the clay located beneath the bottom of the tunnel must be stiff enough to withstand the downward pressure exerted by the footings. This condition was satisfied north of the river, where liner plate method was used. On the other hand, the clay at the south of the river was too soft to support the heavily loaded footings. Inadequate support of the footings of the arch ribs in a liner plate tunnel could cause not only tunnel roof failure but also adjacent buildings collapse. Therefore, in the loop district, shield method was employed in spite of the higher construction cost. Figure 2 shows cross sections of the existing station and tunnels built in 1940's. The outside diameter of the individual tubes is about 7.5 m and the minimum horizontal distance between them at the mid-height of the tunnels is about 2′–8″ (Figure 2).



a) Station cross section (triple arch)



b) Tunnel cross section (circular arch)

c) Existing subway station (triple arch)

Figure 2. Existing tunnels.

3 SUBSURFACE INVESTIGATION

The subsurface investigation for this project consisted of performing geotechnical borings with standard penetration tests and cone penetration tests with shear wave velocity measurements. Laboratory tests were also conducted on representative and undisturbed samples of soils which consisted of index tests, unconsolidated-undrained triaxial tests, consolidated-undrained compression and extension triaxial tests in conjunction with laboratory shear wave velocity measurements.

The subsurface soils consist of an upper layer of miscellaneous sandy fill underlain by layers of glacial deposits above the limestone bedrock. The glacial deposits are Blodgett, Deerfield, Park Ridge, Tinely and Valparaiso. The three layers of Blodgett, Deerfield, and Park Ridge are predominantly clay ranging in strength from soft to stiff. The top of the clay is located at a depth of about 4.5 m below the street surface, approximately at the level of the Lake Michigan. The existing tunnels and the proposed construction are predominantly in Blodgett layer.

Blodgett stratum is characterized as very soft to soft silty clay with trace of fine sand and gravel and shale and mostly shows a variable distribution of water content. Red and Blue line subways were constructed in this layer and the proposed connection at Block 37 is expected to be constructed entirely in this stratum. The upper part of the Blodgett layer to a depth of about 1.5 m is desiccated clay crust and much stiffer than the underlying soft clay. However, the clay crust is very variable in thickness and strength and therefore, the entire stratum was considered as Blodgett layer with softer clay characteristics. The groundwater table is approximately at the top of the Blodgett stratum.

4 DESIGN CONSIDERATIONS

Minimum surface settlement, minimum disturbance to the street traffic and minimum impact on the operation of the existing tracks were the main requirements for the design. The critical design criteria consisted of strict limitation of surface subsidence due to tunneling at the location of existing building foundations. Based on these criteria the angular distortion at these locations can not exceed 1/1000, and no surface point shall subside more than 12 mm below its pre-construction location. Presence of important high rise historical buildings supported on shallow spread footings in the zone of mining influence dictated such strict settlement criteria.

The state of the art geotechnical testing and interpretation program has been carried out by Northwestern University to define the behavior of different Chicago clays at the project site. The type of ground in which the tunnels will be mined is classified as soft clay with very low strength characteristics. Different ground improvement techniques and various pre-support methods were considered in initial phases of design. Ground improvement approaches such as jet grouting and ground freezing were excluded due to their unsatisfactory performance in similar geological settings and also their incompatibility with the densely distributed utilities and other site constraints.

Use of forepoling and interlocking pipe arch techniques as pre-support methods were studied in detail. The interlocking pipe arch concept was selected for the final design to achieve the above described surface settlement criteria. An interlocking pipe installation technology was recommended to minimize installation induced soil disturbance, excess pore water pressure and surface settlement. This technique combines drilling with continuous controlled jacking and auguring.

To limit the face extrusion and its resulting ground deformation and to increase the excavation stand up time, fiberglass face reinforcement bars were designed as an element of the pre-support system. The reinforced core in the excavation face reduces ground deformations including extrusion, convergence and pre-convergence components. The design provides either interlocking pipe support or fiberglass reinforcement in all the exposed excavation surfaces including crown, sidewalls, invert and face of all excavations.

The sequential excavation method has been selected as a means of progressively excavating and supporting the ground. The longitudinal tunnels and cross adits are to be excavated with two headings in accordance with the specified sequences designed to stabilize the ground in limited excavation rounds. The excavation sequences of specified round lengths consist of application of steel fiber-reinforced flashcrete to newly exposed surfaces, installation of lattice girders at predetermined intervals, and application of fiber reinforced shotcrete support with variable thicknesses depending upon final opening dimensions.

Due to its particular geometry, the State Street tunnel BD is to be excavated full face. The proposed steel fiber reinforced shotcrete of initial and final liners are combined and the lattice girder is not used as a support element. For shorter connections in State and Dearborn streets a horizontal secant concrete cylinders concept is used as pre-support system. The design of State Street tunnel BD and the longitudinal tunnel were adapted to accommodate the presence of the paid transfer tunnel and the abandoned freight tunnel. The design of the longitudinal tunnel at Dearborn Street was modified to account for the presence of the active and abandoned freight tunnels in Randolph Street.

The final tunnel lining will be designed to carry the combination of full overburden and hydrostatic groundwater loads as well as surcharge loads. Due to the varying tunnel shapes and short tunnel lengths, it is proposed to use high quality shotcrete for the permanent tunnel support lining.

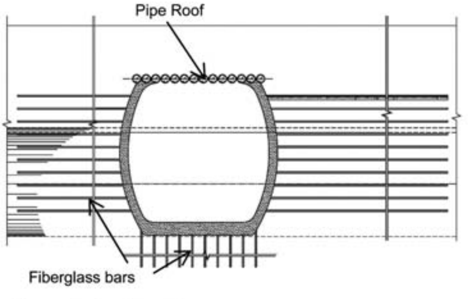
Because of very low assumed permeability of the ground and small expected water head, combination of relatively thick shotcrete initial and final liner with appropriate mix design is considered sufficient to provide acceptable watertightness for the new tunnels. Fuko grouting hoses at construction joints and intersections can be used for repairing the possible leaks.

Monitoring of surface movements will be conducted with series of surface settlement points installed in a series of arrays perpendicular to tunnel centerline across the expected settlement trough. In addition, inclinometers, extensometers and piezometers will be installed to detect horizontal and vertical subsurface movements and ground water level. Total station technology will be used to determine buildings deformation in the zone of influence of tunneling operation. Monitoring points and tiltmeters will be installed to observe any building movement. In-tunnel monitoring will primarily comprise convergence monitoring of the new and existing tunnels and deformation and stress measurement of liner using strain gauges and concrete and soil pressure cells.

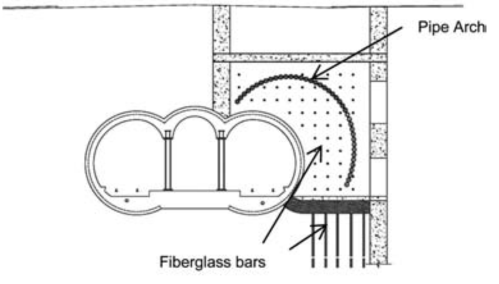
5 CONSTRUCTION ELEMENTS

The construction elements and the corresponding sequences are as follows:

- Building a stairway tunnel under the center arch of the existing tunnel (Figure 4),
- Reinforcement of existing tunnels by erecting new columns, beams and temporary columns (shoring) and installing tie down anchors for both existing Red and Blue line tunnels,
- Installation of 0.6 m diameter interlocked steel pipe roof system and filling them with cement grout (Figure 3),
- Construction of two cross adits under State and Dearborn Streets based on prescribed excavation sequence and support of face, side walls and invert by steel fiber reinforced shotcrete and fiberglass bars (Figure 3),
- Construction of the new expanded tunnel based on prescribed excavation sequence and installation of initial steel fiber reinforced shotcrete and lattice girder support (Figure 4),
- Completing final support system consisting of a combination of cast in place reinforced concrete framing and steel fiber reinforced shotcrete and lattice girder allowing for opening arches at breakthrough points,
- Construction of Tunnel BD at State Street side using interlocking steel pipe arch and fiberglass face reinforcement techniques based on prescribed excavation sequence together with steel fiber reinforced shotcrete as the final liner,
- Connection of track BD at Dearborn Street to the new expanded tunnel at the cross adit location using a short reinforced concrete box structure,

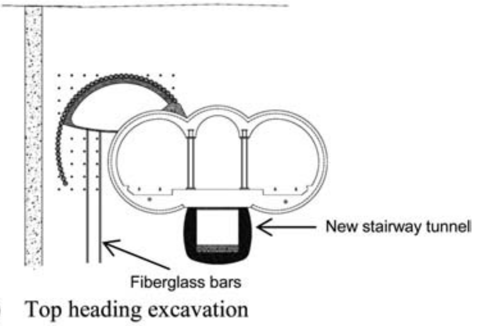


a) Longitudinal section

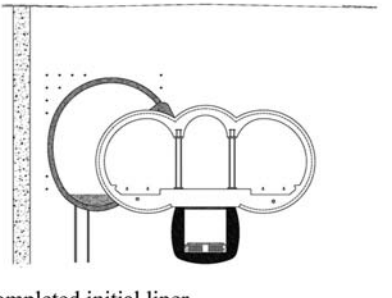


b) Transverse section.

Figure 3. Cross adit sections.



a) Top heading excavation



b) Completed initial liner

Figure 4. New expanded tunnel (during construction).

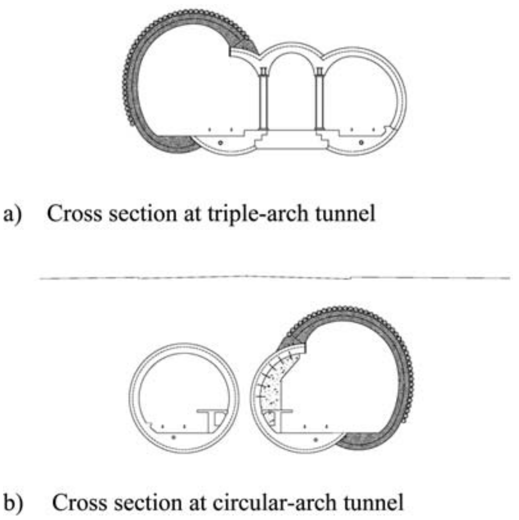


Figure 5. New expanded tunnel (after construction).

- Connection of Track AC at State Street and Dearborn street with two new expanded tunnels at intersection points using horizontal secant concrete cylinders pre-support system and steel fiber reinforced shotcrete and lattice girder as the final liner,
- Backfilling both cross adits based on prescribed backfilling schedule,
- Breakthrough sidewalls of existing tunnel (Figure 5),
- Transferring the loads of removed beams and columns of existing tunnels to new framing systems by means of jacking.

The excavation of both expanded new tunnels will be performed in two stages of top heading and bench. Maximum distance between faces of top heading and bench will be 9 m. The more desirable full face excavation cannot be applied due to constructability constraints which would require hand mining or the use of a movable platform for mechanical excavation. The Tunnel BD at State Street side will be excavated full face in one stage.

6 NUMERICAL ANALYSES AND MODELLING

The three layers of Blodgett, Deerfield, and Park Ridge along with the miscellaneous fill were modeled in the finite element analysis. The Hardening Soil (H-S) model (Schanz, 1999) was used for modeling Blodgett, Deerfield and Park Ridge strata. H-S model is an advanced model with stress-dependant soil stiffness using the following three different input stiffnesses: the triaxial loading stiffness (E_{50}), the triaxial unloading stiffness (E_{ur}) and the oedometer loading stiffness (E_{oed}). The limiting states of stress are defined by the friction angle (ϕ), cohesion (c) and dilatancy angle (Ψ). The relatively simple Mohr-Coulomb model tends to overestimate soil dilatancy. For miscellaneous fill, Mohr-Coulomb soil model was used in the analysis. Material properties used for analysis of Block 37 tunnel connections were obtained from Geotechnical Baseline Report (Patrick Engineering, Inc., 2005) and hardening soil parameters developed for this project at Northwestern University (Finno, 2005).

The excavation and support of various elements of the project such as the extended stairway tunnel, cross adits, expanded tunnels and tunnel BD have been modeled according to the construction stages of the existing and new tunnels using PLAXIS software (Brinkgreve, 2002). As an example, the stages used to model the construction of expanded tunnels are as follows:

1. Initialization of stresses. In this stage the soil layers and ground water table are defined and vertical and horizontal stresses are generated before the construction of existing tunnel.
2. Construction of existing tunnels (initial liner). Construction of existing tunnels, which took place in 1940's, was modeled by creating their geometry as closely as possible and then modeling the excavation using undrained analysis. The initial liner consisting of a steel segmental liner was installed in this stage. This liner was modeled using a structural beam element. The beam element section properties were obtained from the existing drawings. To model the effect of compressed air used during the construction of these tunnels, the groundwater pressure inside the excavated tunnel was kept active, since the pressure of air was originally designed to compensate the existing groundwater pressure during the excavation of these tunnels. Moreover since the concrete final liner was installed at a considerable distance from the face of excavation, 100% of excavation induced stresses are released at this stage. A uniform surcharge of 12.5 kPa was applied at ground surface at this stage representing traffic live load. This surcharge was kept active during the following stages.
3. Construction of existing tunnels (final liner). In this stage, the concrete final liner for the existing tunnels was installed and the groundwater pressure inside the tunnels was deactivated to model the removal of compressed air.
4. Consolidation for 65 years. This stage was added to create the existing state of stress in soil layers, as closely as possible, before the new construction for Block 37.
5. Pre-construction support. Prior to new construction, the existing tunnels were supported by installation of tie-down rock anchors. The displacements

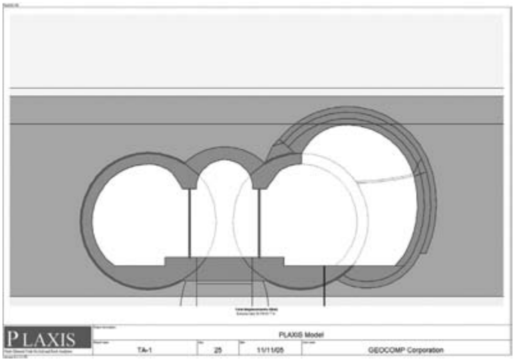


Figure 6. PLAXIS model used in the analysis.

from previous stage were set to zero at this stage, which marks the start of Block 37 construction. Undrained analysis was used for this stage.

6. Construction of new arch (top heading-1). In this stage the top heading of the expanded tunnel was excavated after activating the longitudinal pipes of the pipe arch. It is assumed that 50% of the excavation induced initial stresses are released in this stage. Undrained analysis was used for this stage.
7. Construction of new arch (top heading-2). In this phase, top heading initial liner was activated (shotcrete and vertical fiberglass bars in invert) and 100% of the initial stresses at the excavation boundary were released. Undrained analysis was used for this stage.
8. Construction of new arch (bench-1). The bench of the expanded tunnel was excavated and 50% of the initial stresses were released. Undrained analysis was used for this stage.
9. Construction of new arch (bench-2). In this phase, bench initial liner was activated and 100% of the initial stresses at the excavation boundary were released. Undrained analysis was used for this stage.
10. Cutting the existing tunnel liner. At this stage final liner was installed and part of the existing tunnel liner was cut.
11. Consolidation for 100 years. A 100-year consolidation was performed to calculate long-term ground movements and stresses in surrounding soil and liner.

The tensile stresses in the existing tunnel liner were checked against the rupture modulus of concrete after applying the appropriate load and resistance factors. The analysis indicates that the surface settlement and angular distortion were within the acceptable limits (Figure 6).

7 CONCLUSIONS

A brief history of tunnel construction in Chicago area in the past century including the complexities and challenges involved was presented followed by an overview of construction activities for development of Block 37 in downtown Chicago. A comprehensive geotechnical investigation has been carried out to identify soil parameters required for the design of the connection tunnels at Block 37.

The methodology applied for selection of optimum construction techniques and their complexities were discussed. The construction method included application of fiberglass bars for face reinforcement and interlocking pipe roof technique to minimize the surface settlement induced by tunneling work.

To verify the efficiency of the applied construction technique, series of finite element analyses were performed to investigate: (a) the initial state of stress in soil due to construction of the existing tunnels, (b) the soil-structure interaction during various stages of construction, and (c) the structural response of the existing tunnels.

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Construction of a large underground station under a station in operation

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ABSTRACT: A four- and five-tier box-shaped underground station, measuring 25 m in width and 420 m in length, was constructed by East Japan Railway Company at a depth of about 25 m immediately under Yokohama Station through which about 2.3 million passengers pass each day, to link the Minatomirai Line with the Tokyu Toyoko Line. The construction work faced various challenges, including excavation through special geology, the high level of groundwater, tunneling for a railroad immediately under a large terminal station, and severe restrictions on construction due to live flows of passengers. These obstacles were overcome by meticulous engineering studies and technical development, as a result of which the underground station was finally completed two months ahead of schedule.

1 OUTLINE OF THE PROJECT

Operated by the Yokohama Minatomirai Railway Company and initiated under the Minatomirai 21 Plan, one of Yokohama City's urban plans, the Minatomirai Line is a new 4.1-km line connecting Yokohama Station with Motomachi-Chukagai Station. The Minatomirai Line directly connects with the 2-km line between Higashi-hakuraku Station on the Tokyu Toyoko Line and Yokohama Station, which was also recently built underground. This Project has reduced the time taken to travel from Shibuya Station, Tokyo, to Motomachi-Chukagai Station to a minimum of 37 minutes. East Japan Railway Company (JR) constructed an underground station immediately under the present Yokohama Station under a contract with the operator (Fig. 1).

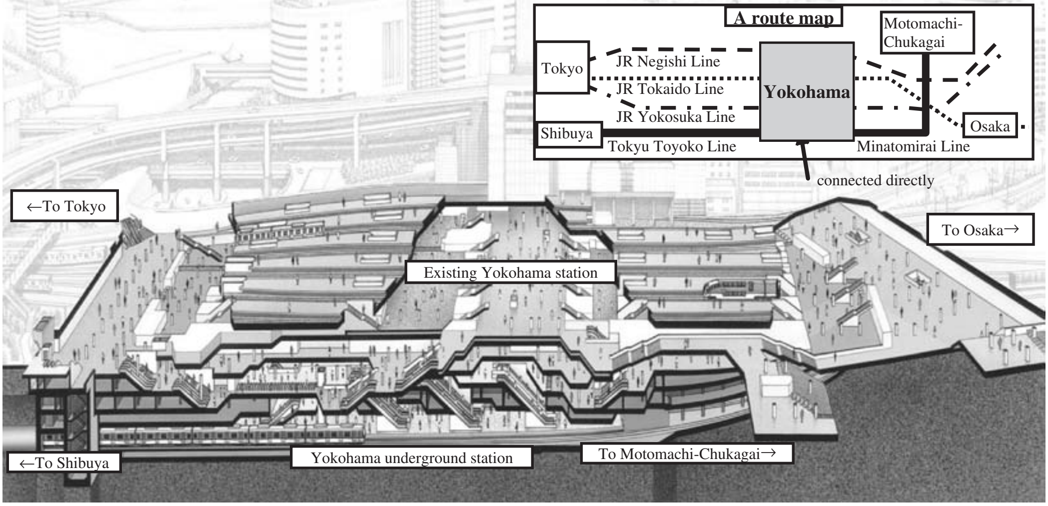

Figure 1. As-built of Yokohama underground station and a route map.

Yokohama Station is used by about 2.3 million passengers every day, serving JR's Negishi Line, Tokaido Line, and Yokosuka Line, the Tokyu Toyoko Line, the Keihin Kyuko Line, the Sotetsu Line and the Yokohama municipal subway through various structures, viaducts and underground tunnels. Yokohama Station is surrounded by station buildings to the east and west, as well as the Aratama River and Katabira River that run north to south.

With no vacant space available above ground, the connecting point of the Minatomirai Line and the Tokyu Toyoko Line was set at about 25 m under the JR Yokosuka Line and the Tokyu Toyoko Line, and a four- to five-tier box-shaped underground station measuring 25 m in width and 418 m in length including the crossing part was constructed.

This paper reports mainly the challenges in terms of geological conditions, working space, available working hours and many other restrictions that were met while constructing this large underground station under a busy, large terminal railway station.

2 AREA SURROUNDING YOKOHAMA UNDERGROUND STATION

2.1 *Geology and underground conditions*

The geology of the ground is very different between the Tokyo side of Yokohama Station and the Kobe side in terms of the depth of bearing stratum, status of geological stratification, and other geological parameters. The Tokyo side ground has good geological conditions, with the Kazusa Group lying between a depth of about 15 m and the ground surface and having an N-value of over 50, whereas the Kobe side ground has a cohesive geological condition down to a depth of greater than 40 m from the ground surface, with an N-value of less than 2. It was a major design and engineering challenge to construct the underground station in such complicated ground.

In addition, a high level of groundwater and the presence of confined groundwater over the entire construction site posed further difficulties for the construction work, including the potential lifting of the underground station structure. Therefore, meticulous design and engineering studies had to be conducted (Fig. 2).

2.2 *Structural outline of the underground station*

The Yokohama underground station for the Minatomirai Line and the Tokyu Toyoko Line is a box-shaped structure measuring about 418 m in length, 25 m in maximum width, and 25 m in height. The dimensional details include 24 m for a vertical shaft, 209 m for the structural shell of the station, and 185 m for the crossing under the JR tracks. The structural shell of the station is located immediately under the track of the Yokosuka Line and the viaduct of the Tokyu Toyoko Line. The underground station is thus composed of five layers: one aboveground level that is integrated with the existing JR Line viaduct and four underground levels, as well as two spans (Fig. 3).

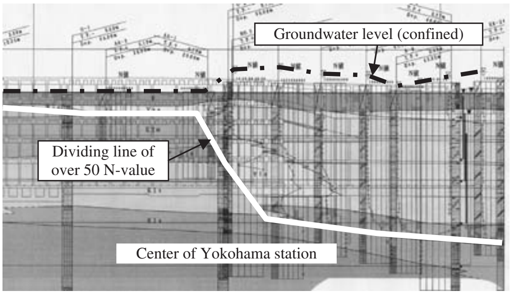

Figure 2. Schematic diagram of Yokohama underground station, geology and groundwater.

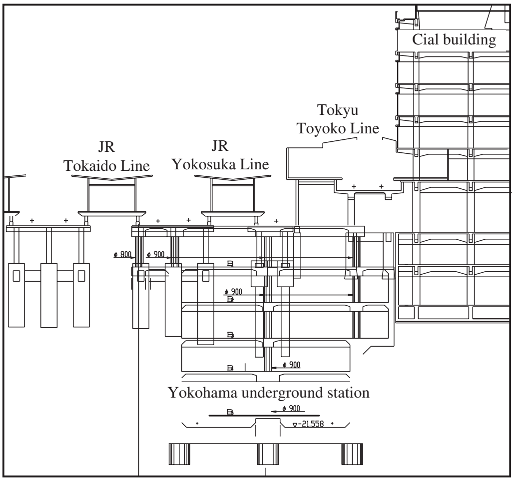

Figure 3. Main body of station – standard section.

In view of the fact that a building is planned to be constructed on the ground above the underground station, specifically a 85-m section from the starting point of the station shell, the steel reinforced concrete construction is adopted for the 85-m section to ensure architectural consistency with the specifications of the future building. The remaining 124-m section was constructed using two different types of construction to reduce the construction period of the underground station although the original plan was to use reinforced concrete construction only. To be specific, steel construction was adopted for the beams and columns of the aboveground part (B0F) and part of the underground level (B1F and B2F), while reinforced concrete construction was used for levels from B3F to B5F.

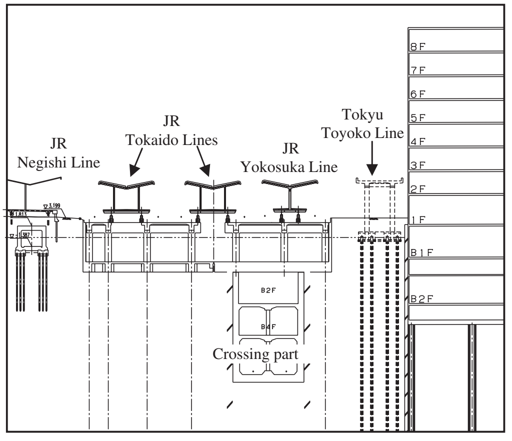

Figure 4. Crossing part – standard section.

The crossing section, which is near the end of the underground station, is a 185-m tunnel that crosses sideways immediately under the existing eight JR lines and connects the main body of the station to the shield-excavated tunnel near Motomachi-Chukagai station.

The structural shell of the crossing section is a reinforced concrete box rigid-frame tunnel, measuring about 14 to 18 m in total width and about 18 to 20 m in height, and with the number of levels and spans varying from 4 and 2 for the station proper to 3 and 1 for the shield-excavated tunnel (Fig. 4).

2.3 *Construction of the underground station*

2.3.1 *Restrictions on work*

Yokohama Station is a large terminal station with some 2.3 million passengers daily, and also operates cargo trains running on a tight schedule. Construction of the underground station faced various restrictions, including that no trains of the existing lines should be forced to reduce speed during excavation. A subway station is generally constructed perpendicular to the direction of the existing track to minimize the influence on the track, but the Yokohama underground station had to be built parallel with the existing tracks since the station buildings and their basements already stand along the existing tracks. Therefore, temporary girders to support train loads had to be built for about 2.4 km in total (283 spans in total). Further restrictions included very short night-time working hours of 3 hours minimum and about 4 hours on average between the last train and the first train.

2.3.2 *Construction procedure*

The Yokohama underground station was constructed as follows, as illustrated in Figure 5.

The existing Yokohama Station, before commencement of the underground station work, had train tracks both on viaducts and embankments. Before excavating

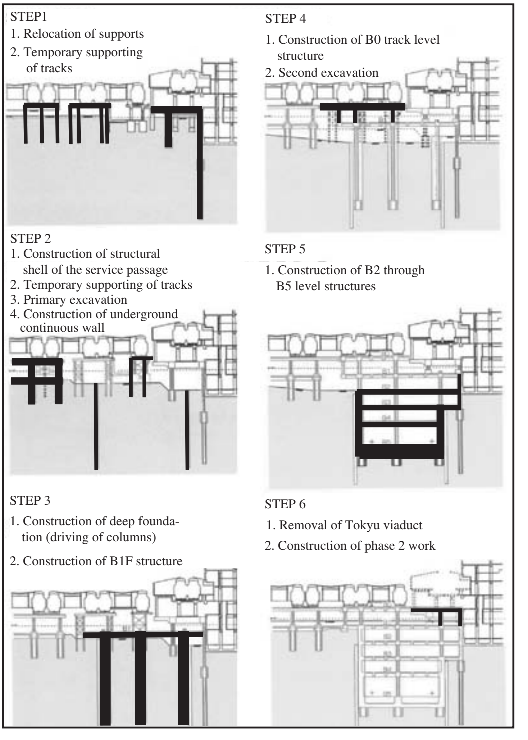

Figure 5. Construction procedure of Yokohama underground station.

the ground, the JR line tracks and the Tokyu Toyoko Line tracks had to be temporarily supported to ensure their structural stability.

[Step 1]
A tentative viaduct was constructed by JR East Japan to support the JR line tracks, while the Toyoko line tracks were temporarily supported by the underpinning method by Tokyu Railroad Company.

[Step 2]
The ground was excavated down to the B1F level of the underground station to create a working space immediately under the existing railroad tracks. Then, the cast-in-situ diaphragm walls were constructed as earth retaining walls to allow further excavation down to the B5F level.

[Step 3]
The ground was excavated down from the B1F level to the B5F level by means of the deep foundation method, and steel pipe columns were constructed from down to up. Then, the structural shell of the B1F was constructed.

[Step 4]
The viaduct to support the JR line tracks was constructed and replaced the temporary viaduct, which was then demolished and removed.

[Step 5]
During construction of the viaduct for the JR line tracks and demolition of the temporary viaduct, the process of excavation and construction of structural shells was repeated to construct the underground station, using the inverted lining method as the main construction method.

The underground station was opened on January 31, 2004, which was the timing when Step 5 was completed.

[Step 6]
From February, the Tokyu Toyoko Line was relocated to a newly constructed tunnel under Yokohama Station, and the former viaduct was demolished. Ongoing construction of the ground level structure of the underground station is the last step, completion of which will mean the completion of the Yokohama underground station.

3 NEWLY INTRODUCED OR DEVELOPED TECHNOLOGIES

The staff repeatedly studied various ways to complete the project within the schedule, and a number of technologies were improved and newly developed to meet the challenge and cope with various restrictions posed by the surrounding conditions and associated engineering difficulties.

3.1 *Development of screwed rotational jack steel piles*

Temporary piles for work girders had to be driven to a depth of about 30 m because of the soft ground and the deep bearing stratum, but as there was only a 2.5-m clearance due to the feeder lines, 17 parts had to be connected to form a single pile. In addition, each pile had to be constructed during just a two- to three-hour period at night when the trains were not running. The screwed steel joint technique was therefore developed and used, as shown in Figure 6. This technique successfully increased the amount of work done each night and reduced the pile construction time. In detail, the conventional method requires 90 minutes to construct a welded joint pile, whereas the screwed steel joint technique can accomplish a single pile in 20 minutes. With the conventional welding joint technique, it would have taken 5 or 6 years to complete all the piles, while the new technique could finish it in almost half the time, or about 3 years.

3.2 *Development of attachable pile cap joint*

Steel pile caps needed to be quickly, neatly and precisely treated to construct lateral girders of temporary viaducts. With the conventional welding connection,

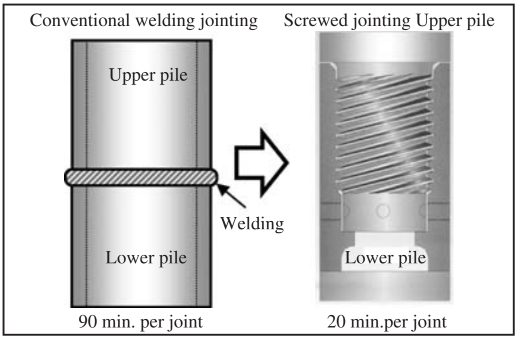

Figure 6. Schematic diagram of screwed steel pipe joint technique.

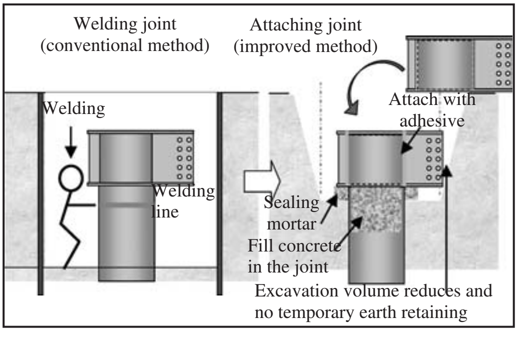

Figure 7. Schematic diagram of attachable pile cap joint technique.

not all the procedures for pile cap treatment could have been completed in just three hours at night. With the screwed technique, however, sufficient precision in the connecting joint was not possible, so a new technique, as shown in Figure 7, was developed. In the new technique, a connecting joint is put over a steel pile head like a cap, the cap-like joint is connected to the steel pile with acrylic resin adhesive filled into the space between the joint and the pile top, and concrete is filled inside the joint to ensure integration. This technique reduced the work load and the volume of excavation, eventually finishing the pile cap treatment work in a single night, down from two nights.

3.3 *Development of the mechanical deep foundation construction technique*

It was necessary to construct main posts for the underground station (2.6 m in diameter and about 30.0 m in length) under limited working conditions with a short overhead space immediately below the existing railroad tracks, and so a deep foundation had to be constructed. However, the conventional approach could not have completed the necessary work safely and smoothly because of the poor working conditions

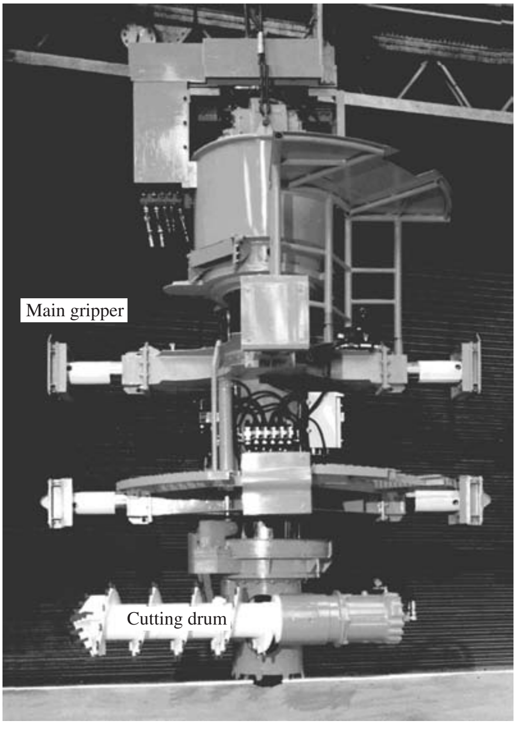

Figure 8. Deep foundation construction machine.

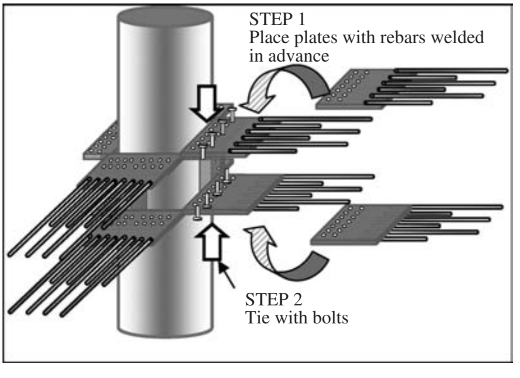

Figure 9. Illustration of connection.

and small number of experienced workers. Therefore, an automatic machine designed to excavate and install liner plates was developed, as shown in Figure 8. This machine successfully allowed safe and smooth construction of a deep foundation in small working spaces.

3.4 *Development of a new technique to connect steel pipes and RC beams*

Steel pipe columns were used for the main columns of the underground station to shorten the constrution period, while reinforced concrete beams were used for those connecting the columns to improve economic efficiency. Since there were about 400 places where steel pipes had to be connected to RC beams, a simple connecting technique was needed. A new technique was developed involving an integrated structure in which diaphragm plates at the connection of the steel pipe columns are tied by bolts to plates with mechanical joint rebars welded in advance. The application of this type of precasting technique to the connections of steel pipes and RC beams successfully reduced the field working hours and greatly shortened the total working period (Fig. 9). Before being put to practice, this new technique was proven to be effective by conducting a scale-model experiment and three-dimensional FEM analysis.

3.5 *Development of the steel element continuous wall*

For excavation for the external part of the western side continuous wall (referred to as "extended part") of the main part of the station, and construction of the structural shell, the original plan was to first construct the main body of the station under B2F, without which no extended part could be constructed in terms of structural stability. However, we were requested by the client to bring the start-up timing forward by two months, so we had to shorten the construction period. We drove angular steel pipes called JES (jointed element structure) vertically while manually excavating the ground, and filled the inside of the pipes with concrete. This work eliminated the need to construct the main structure of the station and allowed construction of the main walls that also serve as temporary earth retaining walls without affecting the structural stability of the underground station. Eventually, this technique shortened the original schedule by three months, and it was applied over an area of some 500 m^2. JES are generally applied horizontally in the construction of a tunnel crossing under a railroad track, but for this project they were used vertically (Fig. 10).

4 CONCLUSION

The Tokyu Toyoko Line tracks were relocated to the underground tracks on January 31, 2004, which allowed use of the Yokohama underground station. Then, the Minatomirai Line was opened on February 1, 2004. The work faced a number of challenges, including various restrictions posed by operation of train services, fluctuating loads of station users and special geological conditions immediately under the existing large terminal station. Strenuous efforts were made to study the conditions and develop new engineering ideas, and the project was successfully completed

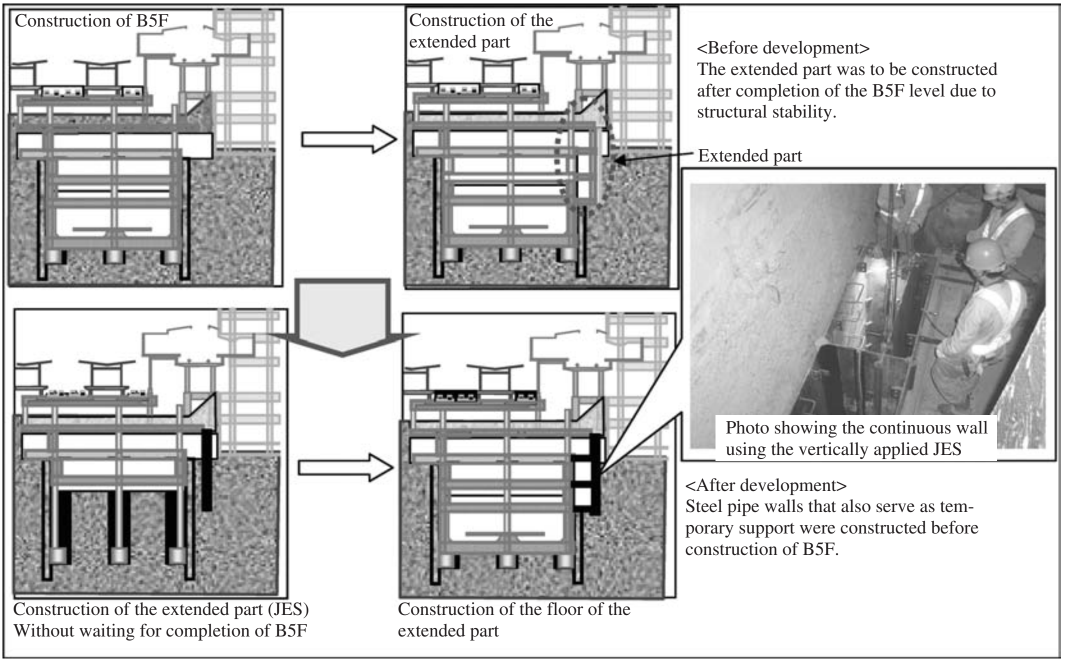

Figure 10. Schedule reduction by construction of continuous diaphragm walls.

about two months earlier than originally planned (March 2004).

The Minatomirai Line has started operation, but some work remains to be done, including deconstruction of the former Tokyu Toyoko Line viaduct and associated construction of the second phase of the underground station. The authors hope that this report may be of help to similar projects in future. Finally, the authors would like to thank all the people involved in this project.

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The next step: The Very Long Tunnel (VLT)

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COWI AS

ABSTRACT: This paper attempts to see into the future of subaqueous tunneling and look at some exciting technologies that are beginning to come together to make possible the efficient underwater transportation of people and cargo over heretofore unheard of distances. Further this paper identifies some of the unique environmental benefits of the Submerged Floating Tunnel concept when combined with underground and commercial and municipal facilities. stations, shops and any other suitable activity needing space.

1 INTRODUCTION

At the beginning of the 20th century the only known ways to construct a tunnel under a waterway were by simple digging, sometimes under air pressure and by a new "high tech" method, just emerging initially referred to as "Sunken Tube Tunnel". During the 20th century over 150 tunnels were constructed by this Immersed Tunneling(IT) method. These tunnels were used for many purposes including rail and highway transport, material transport, and the subaqueous crossing for various utilities such as water, sewer, petroleum etc. The Immersed Tunnels are prefabricated in sections 100–200 m. long, sealed by bulkheads at the ends, floated to the tunnel site and lowered into a trench whereby they are joined hydraulically and later backfilled.

Another idea for underwater tunnels had been around for nearly 150 years but was never seriously considered until recently. The idea of a tunnel with a vertical alignment above the bottom of the waterway and either floating with some form of stabilizing support, or simply supported on underwater piers in the form of an underwater bridge. This is what is termed the Submerged Floating Tunnel (SFT). This idea was originally proposed in 1865 for an underwater bridge crossing the English Channel, by a British Naval Engineer, Sir Edward James Reed.

In the 1980's and 90's several crossings of this kind were seriously considered and preliminary designs were developed, notably in Norway (Hogsfjord), Sicily (Strait of Messina), Italy (Lake Lugano) and others in Japan.

As a result of the increasing use of the immersed tunnels for underwater crossings all over the world and the new found interest in the SFT Concept, in 1987 at the World Congress in Manchester, UK, ITA established Working Group 11 "Immersed and Floating Tunnels" to foster these tunneling technologies internationally.

Since then WG 11 has presented many technical papers and has also published two editions and most recently an on-line version of the "Immersed and Floating Tunnels State of the Art Report".

This paper is now intended to take these existing technologies for underwater tunnels and suggest means by which they can be extended perhaps by a factor of 100 or more to serve the developments in long distance transportation in MAGLEV.

We name these extended tunnels "VLT", (Very Long Tunnels)

An extension of VLT is very briefly mentioned at the end of this paper, the Trans Ocean Tunnel (TOT) The combination of land based tunnels and submerged floating tunnels facilitate the use of underground space for a long list of purposes, such as underground parking, service stations, shops and any other suitable activity needing space.

2 EXISTING TECHNOLGY

2.1 *The immersed tunnel*

The immersed tunnel method is a well developed technology of great versatility. None of the existing tunnels are alike. Each is designed to serve a set of specific requirements for interior space, type of throughput, environmental conditions, construction constraints, ventilation, structural strength, seismic loads, accidental loads, etc. Because of this, there exists a wide range of available design and construction experience on which to draw in moving to the development of the as yet untried SFT.

There are, for example, a few basic structural configurations used in immersed tunnels. The most prevalent, particularly for vehicular traffic, is the rectangular concrete box section. This has been widely used in Europe and other parts of the world. In the United States where the technology started however, various forms of single and double steel shell structurally composite sections largely became the norm. In these sections, the steel shell both acts as a structural composite medium with an interior concrete load carrying lining and as a waterproof enclosure..

The construction of these tunnels is a shipyard operation with steel shell elements being assembled from prefabricated modules into long single or double tubes. Bulkheads are fitted to the ends and the element can either be end or side launched or it can be floated out if assembled in a drydock. The interior concrete structure is then largely constructed while the element floats at a jetty. The foregoing methodology is particularly well suited to the development of the SFT.

2.2 *The Submerged Floating Tunnel*

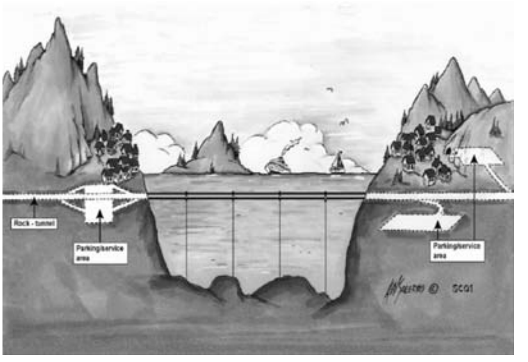
It is now time to describe the SFT as this is the bridge between the immersed tunnel and the Very Long Tunnel (VLT) the subject of this paper. The SFT takes the immersed tunnel and raises it out of the mud, so to speak. The immersed tunnel has an important advantage over the excavated tunnel in that its depth need only be a few meters below the bottom of the waterway. A shield driven tunnel, for example, must be at least a diameter below the stable bottom putting it perhaps ten meters deeper or more. For a rail tunnel with flat gradients of say 0.015 this will increase the total length of the tunnel by (10/.015)2 = 1,333 m.

Take the example of a crossing under a fjord where the depth might be 500 m. Given the same navigation depth requirements as for the previous example, the SFT would only be as long as the immersed tunnel, but in this case the immersed tunnel would be outside of the range of feasibility for IT's and a bored tunnel would be say (530/.015)2 = 71 km longer!

At such a depth to cross, it could be anchored directly by steel tethers attached to bottom anchors spaced along the tunnel alignment.

2.3 *Design of the SFT*

As for the IT, almost any configuration can be adapted for the SFT. The structure can be solely concrete, solely



steel, or a steel/concrete composite. It can be multi-lane or multi-track. It can be rectangular or it can be circular. Whatever the configuration, as the depth is made greater the structure increasingly favors a circular cross section for efficient design. Tethered SFTs must float with sufficient positive buoyancy to maintain stable tension in the tethers under all loading conditions. At greater depths, the concrete becomes too much of a dead load so that steel alone may be used. Exterior flotation material may be used to augment positive buoyancy.

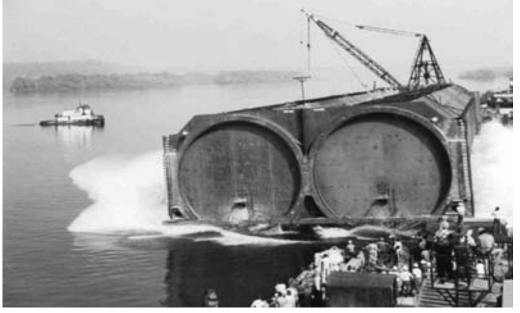
Dynamic loads caused by waves, swell, wave drift forces, vortex shedding or varying currents can cause trouble if these forces are not well understood and properly accounted for. Often physical modelling can be helpful directly or can be used to calibrate and verify software for appropriate modelling of such phenomena. Such model testing and evaluation of the test results with particular emphasis on the slowly-varying motions have been done in the Høgsjord project. Based on the present state of the art, circular cross-sections are favoured also from a hydrodynamic point of view.

The terminal ends of the SFT where the tunnel enters the rock or the ground will require special design to limit the motion without focusing load, especially if seismic forces must be taken. Potential lateral, vertical, and longitudinal motions must be taken into account.

2.4 *Construction of the "unsinkable" SFT*

As this discussion will eventually lead us to the next step, the VLT, let us say that we are going to build a tethered, steel shell SFT to cross a 2000 m wide waterway 400 m deep at the center. This SFT will carry three tracks for electric trains. The signal system will be operated so that no more than one train can be in the tunnel on two of the tracks at any one time. Further the tunnel will be designed so that it will not sink and self-destruct if it is breached and filled with water.

The bottom of the water channel is soft and has a low bearing capacity so it must be improved by piling or simply gravel replacement of the soft strata. Large gravity anchors will be used. The elements will be



Example of immersed tunnel of steel.

An approved alternative for SFT and possible anchoring system for (VLT).

200 m long and the anchors and double tethers (on each side) will have a longitudinal spacing of 50 m or four sets per element located so that they miss the joint areas.

Each of the ten elements is constructed on side launching shipways and all are straight as drainage will all be toward one end. A 2 meter thick layer of syntactic foam (made of glass microballoons that will not crush or deteriorate in water pressure) covers the circumference of the tunnel except at the joint areas providing reserve flotation capable of supporting the tunnel if the interior floods. For the SFT to be unsinkable it must have sufficient flotation capacity to allow the entire running tunnel tubes to flooded while maintaining a nominal tension in the tethers.

The joints are designed and the elements are connected very much using the well-proven method used for immersed tunnels. in that an initial seal is first made to seal the joint area so that the water pressure can be released and the full hydrostatic force at the other end of the element being placed can compress the gasket (GINA) and permit a full structural connection to be made.

2.5 *Tethers and anchorages, offshore Technology*

Over the last few decades offshore platforms have been constructed and operated in deep waters, such as in the Gulf of Mexico, the North Sea and off Brasil.

Vast experience has been gained regarding forces exerted on offshore structures by waves and current, the motions induced by these forces and on the hydroelastic interaction between the forces and the motions. Better understanding of phenomena and sources for slow-varying motions has been acquired. This understanding is of vital importance for the safe design and configuration of an SFT with its anchorages.

The Tension Leg Platform (TLP) with its tethered connections to the sea bottom has in particular paved the way for the use of this technology also for structures like the SFT. This applies both to the design and installation of the tethers and their anchoring elements on the sea floor.

For relatively shallow SFT's with depths in the order of 100 m. cable or pipe tethers may be suitable but as the depth gets deeper, the tethers themselves must be more sophisticated. The offshore oil industry has led the way in this respect in the design of tethers for their Tension Leg Platforms (TLP's). Special pile anchorages and floating hollow tethers have been used offshore. This technology is well developed and should be adaptable and suitable for use for SFT's and VLT's.

2.6 *Tunnel saddles*

The method used to connect and fine tune the tethers to the tunnel structure is an area that still requires a great deal of study. The configuration of the tethers themselves and how they attach to the tunnel and transfer huge loads requires special attention in the structural design of the tunnel itself.

A limitation on the use of the SFT is the depth of water. At some point the tethers become too long and laterally flexible allowing unacceptable deflections. It is doubtful that a completely stable SFT can be built in more that 1000 m. of water depth. Unless measures as described to provide enough flotation in the SFT to prevent its destruction in the event of accidental flooding, such a tunnel would be high insurance risk.

For Høgsfjord it was found that, where the tunnel would arch between the abutments, it would be too vulnerable to slowly-varying wave/current forces, and concluded that tethers with sufficient high lateral stiffness was preferable.

2.7 *Stability of tethers:*

The tethers can be unstable due to varying tether forces (so-called Mathieu-instability) also for relatively short tether lengths. The key thing here is to configure a tether system with high enough stiffness both vertically and horizontally. A combination of inclined and vertical tethers every second pair of tethers might be one interesting possibility.

3 THE VLT

3.1 *The very long SFT + MAGLEV + Vacuum = the VLT*

Say you want to cross the Irish Sea between England and Ireland. It is some 100 km wide and less than 200 m deep. How would one do this. Vehicles are out, but trains maybe...

One concept that has been given some attention recently, is the building of a long tunnel and keeping it at a full or partial vacuum. By using frictionless MAGLEV propulsion, theoretically, trains could travel a very high speeds of say 1000 km/hr or faster in such a vacuum and make the 100 km crossing in ten minutes or less allowing some time for acceleration and deceleration.

MAGLEV or Magnetic Levitation of course is the technology now being used to propel trains at great speed by magnetically raising them above what in effect is the stator part of an alternating current motor. By so doing there is only air resistance to overcome. By removing most or all of the air, the train can operate at very high speeds with little energy usage. Most of the energy used is due to resistance losses in the MAGLEV "rails".

So if we take the SFT example we developed above, build the special termini with airlocks and vacuum pumping stations, provided MAGLEV trains equipped with self-contained breathable air/oxygen and assured a very stable precisely aligned leak-proof tunnel, a new international travel system will have been created.

3.2 *Things to consider*

Of course it is not quite as simple as it sounds. For these tunnels to operate safely and comfortably, their alignment must be maintained to precise tolerances. This is not easy to do in open ocean conditions with storms, waves, currents, varying water densities and the weight of the trains themselves acting to displace the tunnel. This favors the idea of going deeper to stay away from the turbulent surface conditions that may prevail. But going deeper requires more structural strength and more weight. Leakage is not an option as any water entering the tunnel will flash into vapor in the tunnel and provide a local pressure that might buffet the train besides that salt would be deposited everywhere and salt is an enemy of electrical equipment.

A long list of problems will have to be solved, among these the amount of electrical power needed, how to produce it and transport it.

The construction and maintenance of the MAGLEV system, the vacuum system, the air-locked train stations and switching yards would be costly and complicated but certainly doable.

For intermediate access and exit to and from the VLT one can imagine intermediate (tethered) submerged stations provided with shafts to a platform above the wavezone. Very much like the existing Tension Leg Platforms. These stations can initially serve for the construction of the tunnel, later for maintenance and for emergency evacuation.

3.3 *Construction time and cost*

Continuing with our 100 km crossing example what will the components of the system be and how long will it take to construct them? At 200 m. per element, there would be 500 elements to construct.

These units would likely be built in shipyards or specially equipped fabrication sites. For easy installation and hookup, most of the electrical mechanical "guts" of each element would be in place when connected into the tunnel so that only the work in the joint would be required.

It is likely then that the fastest any element could be constructed would be 3–6 months. Based on the present day industrial practice for steel tunnels at shipyards, we assume a learning curve for each shipyard so that the average production speed would be 4 months per element, 500 elements would require 2000 shipyard months. To complete the production in say 10 years would take $2000/120 = 17$ shipyards working continuously.

The cost of such a structure would be high compared with the cost of present day tunnels, however these tunnels may be a possibility to help solving future transport problems.

3.4 *Safety*

3.4.1 *Collision from vessel*

While the chances of such a collision should be very low since the SFT and the VLT are essentially exposed to the danger of a ship sinking or a submarine collision, it may not be possible to design a tunnel to absorb such an impact. However small ships would not be a problem given the mass and strength of the tunnel. It is noted that the current practice in designing immersed tunnels provide for accidental loads by sunken ships that could easily be carried by the VLT. Especially if constructed to be unsinkable. Furthermore, it is entirely possible to replace a damaged or destroyed section of tunnel underwater given that the design has provided for a stable flooded tunnel.

If an SFT/VLT is not capable of floating in a flooded condition it will of course then be lost by collapsing to the ocean floor.

3.4.2 *Terrorism*

For bored or immersed tunnels the danger of terrorism is from the inside. For SFT or VLT the outside is also exposed. The danger of terrorist is always present, any underwater tunnel could be a target with the larger, longer tunnel being a bigger "statement" . As the case of airlines, security will be the first countermeasure. However the use of interior foam for flotation can also be designed to absorb explosive energy so that even though a car and train be destroyed, there may be a way to design and prevent the breaching of a SFT or VLT tunnel.

4 ADVANTAGES OF THE SFT/VLT

The SFT can be environmentally attractive as they put roads or railroads out of sight. This is especially beneficial in areas of natural beauty where congested roads or noisy railroads along lake shorelines can be a real blight.

In certain cases, since the roads entering shoreline towns are entirely hidden underground, they might be connected to parking garages and shopping malls also underground and hidden.

These tunnels save fuel/energy by keeping tunnel alignments flat. Steep grades in conventional tunnels require heavy fuel consumption, especially for cargo trucks.

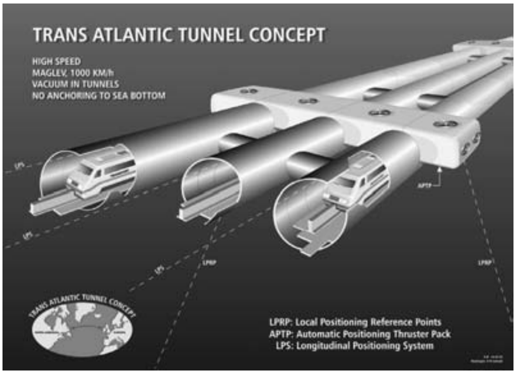
Of course, the proposed combining fast transport with very long tunnels has many potential benefits. While these benefits and technical breakthroughs are very attractive, in the end their costs must be justified by the traffic they will carry.

4.1 *Preliminary conclusion for Submerged Floating Tunnel*

The SFT is a viable environmentally friendly alternative method for underwater transport both vehicular and rail as well as other pipeline, conveyor, or services that might be desired. It is an extension of the immersed tunnel technology with new aspects and engineering problems to solve. No SFT has yet been constructed. This may be because all the SFT's put forward so far are vulnerable to collapse if flooded, but as indicated above, SFT may be made unsinkable. SFT's are more costly on a unit basis of cost per kilometer than tunnels that are excavated but may make up for that by shortening the tunnel significantly.

4.2 *Preliminary conclusion for Very Long Tunnels*

The VLT is an extension to the SFT. The idea of a high speed MAGLEV railroad within a vacuum tunnel seems very novel and interesting. It would only seem to justify itself for significant distances of say more than 50km. Such a tunnel would seem to take many years to amortize its cost and the energy involved in its construction. However with the industrial base in place to produce such tunnels and efforts to standardize components, it might well turn into a wave of the future with longer and longer subsea tunnels eventually transitioning from VLT's to Trans ocean -Tunnels (TOT's)



A futuristic example of a Trans Ocean Tunnel

40 years ago when containerized cargo began to appear in ports, who could have visualized the huge ships carrying 10–15,000 standardized containers of present day?

4.3 *Some thoughts on Trans Ocean Tunnels (TOT)*

Connection between continents may be called for in future for reducing the use of energy in transport of people and goods.

Combination of all kinds of tunnels and tunneling technology would be required, soil and rock tunnels, immersed and submerged floating tunnels and completely new technologies and solutions.

This would be one of the big challenges of the future in the tunneling industry and preliminary investigations and research should start now.

5 HISTORY AND BACKGROUND

5.1 *Brief history*

In 1865, Sir Edward James Reed, MP for Cardiff put forward the idea of crossing the English Channel in a tunnel suspended on top of caissons. The concept did not win much support, as members of Parliament feared this to be a possible route for invasion of the British Isles.

Recently , a number of studies have been carried in Italy, Japan and Norway. One of these studies, for the Høgsfjord Project in Norway was taken as far as being approved for final engineering design by the Public Roads Administration. This project brought forward four separate concepts, three as concrete tunnels and one alternative in steel as the main construction material. The anchoring methods were by pontoons on the surface or as tethers to caissons on the sea bed.

5.2 *Publications and Conferences*

The following is a summary of recent SFT publications, project studies and conferences , this list is by no means complete, but will illustrate the interest this special structure has received .In addition to this list a large number of presentations have taken place in conferences and seminars over the last 20 years or so.

1973 Vallavik-Bu, Norwegian Public Roads Administration (NPRA), reports
1984 Messina Strait
1989 AIOM Congress , Napoli, Italy
1990 Wegen naar de toekomst (Roads to the future) Robbert and Rudolf Daas, publ.Tirion, Baarn, NL
1996 First International Conference on SFT, under the auspices of the NPRA
1996 FEHRL Report on Analysis of SFT concept
1997 ITA, Working Group no.11, State of The Art Report
1999 Strait Crossings Working Group, ITA, NPRA
2000 Seminar Varenna, Italy, Ponte di Archimede, NPRA
2002 First US workshop on SFT, AUCA
2003 The elevated immersed tunnel James Felch, WCT, Amsterdam, 2003
2004 Discovery Channel, Extreme Engineering, Program: “Trans-Atlantic Tunnel”

Some examples of Project studies known to the authors, there are likely other studies not recorded below:

Vallavik-Bu, NPRA, Norway
Høgsfjord Project, NPRA, Norway
Strait of Messina, Italy
Lake Lugano, Switzerland
Funka Bay, Hokkaido, Japan
Lake Mjøsa Crossing, NPRA, Norway
The Storfjorden Crossing, NPRA, Norway
Lake Washington, Washington State , USA
Lago di Como and Magiore, Italy

6 OVERALL CONCLUSIONS

SFT, VLT and possibly TOT will be challenging future opportunities for cooperation within the whole tunneling industry. ITA may be the most important forum for these exciting possibilities.

Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina & Romancov (eds)
© 2007 Taylor & Francis Group, London, ISBN 978-0-415-40807-3

Planning and construction of major infrastructure works in Croatia

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ABSTRACT: The paper presents recent accomplishments in tunnel construction in Croatia, which was part of a much bigger undertaking – development of national motorway network. Over the last decades Croatia made large investments in order to construct the planned 1500-km of motorways. The total length of motorways and semi-motorways currently operated in Croatia amounts to 1020 km, half of which were constructed over the past 5 years. Further works comprise extension of motorway network, as well as rehabilitation of older sections. Development of highway network required construction of a large number of motorway structures. Overall 55 tunnels will in the end provide for little over 86 km of Croatian motorways, including two longest tunnel structures in Croatia: Sveti Rok tunnel (5670 m) and Mala Kapela tunnel (5780 m) completed in 2003 and 2005, respectively.

1 INTRODUCTION

The last decade is marked by extreme growth of construction in Croatia, mainly in terms of intensive development of motorway network. Numerous structures were build to carry motorways across wide rivers, large valleys or deep gorges, cut through mountains, or simply provide facilities for safe and comfortable transport.

Motorway network construction is currently the most important development project in the country. After gaining its independence in 1991, Croatia was faced with insufficiently developed road network.

The strategic significance of the construction of motorways for the Republic of Croatia was soon recognized (Radic et al. 1999) with the long-term development programme of the Croatian road network outlined in Road Network Development Strategy (Banjad et al. 1996) adopted by Croatian Government in 1996 and Croatian Parliament in 1999 (Fig. 1).

Subsequently the construction gained speed and currently the total length of motorways operated in Croatia amounts to 1020 km (www.huka.hr), half of which have been constructed over the past five years. It is planned to construct the total of 1501 km of motorways, and the works are advancing as scheduled (www.hac.hr). Figure 2 indicates the current stage of completion of Croatian motorway network.

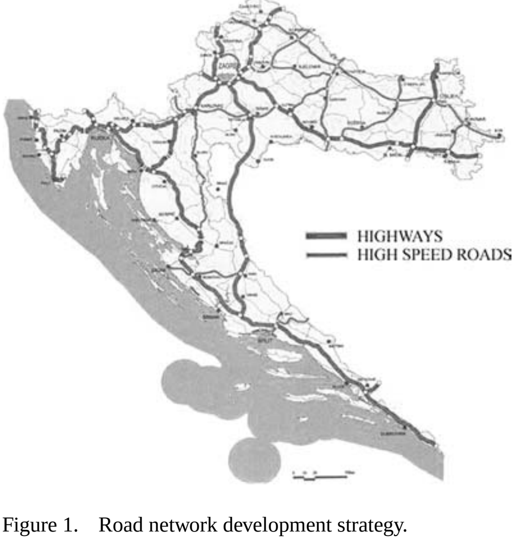

Figure 1. Road network development strategy.

The Croatian motorway network may not seem long, but it is worth mentioning that Croatia already has more kilometres of motorways per 100 000 citizens than UK, Ireland, Greece or Italy.

Development of motorway network required construction of a large number of motorway structures

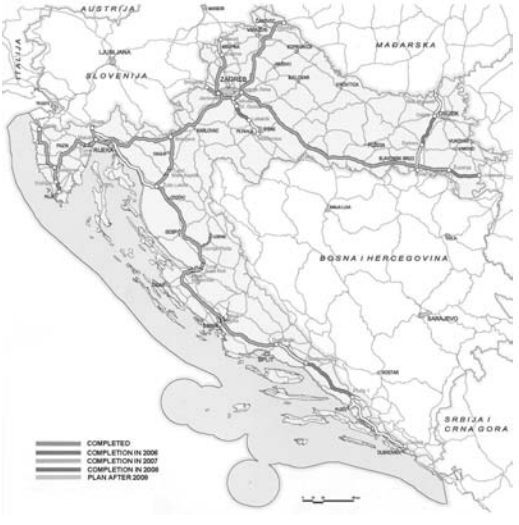

Figure 2. Motorway network in croatia (2005).

Table 1. Number and length of tunnels on Croatian motorways (Mlinaric et al. 2005).

Motorway	No. (pcs)	Length (km)
A1: Zagreb-Bosiljevo-Split-Dubrovnik	22	42.28
A2: Macelj-Zagreb	6	5.31
A4: Goričan-Zagreb	2	2.18
A6: Bosiljevo-Rijeka	12	10.03
A7: Rupa-Rijeka-Zuta Lokva	12	21.21
A8: Kanfanar-Matulji	1	5.06
Total	*55*	*86.08*

(Radic et al. 2006): tunnels, bridges, viaducts, overpasses, underpasses, wildlife crossings etc. For instance, 292 motorway structures account for 19% of the 380-km long Zagreb to Split motorway route. Large-span bridges and long tunnels are the most complex and most expensive part of the network.

2 TUNNELS

2.1 *General*

Overall 55 tunnels (refer to Table 1) will in the end provide for little over 86 km of a total of 1501 km long Croatian motorway network. Two structures stand out – Sveti Rok Tunnel, with total length of 5670 m and Mala Kapela Tunnel with total length of 5780 m, which is the longest tunnel in Croatia, and these shall be described in more detail.

Figure 3. Bisko tunnel.

2.2 *Current works*

Currently, work is in progress on several tunnel structures on Zagreb to Rijeka motorway, Zagreb-Macelj motorway, as well as on motorway sections in the most southern parts of Croatia, which provide for the motorway extension from Split to Dubrovnik.

Most of the structures are constructed as twin-tube tunnels, except on the Rijeka to Zagreb motorway where the current works comprise addition of the second tube to already existing two-way traffic single tubes.

We shall mention only two tunnels among many structures currently under construction: Bisko Tunnel and Strazina Tunnel.

Bisko Tunnel shown in Figure 3 is a twin-tube tunnel with the total length of the left-hand side tunnel tube of 493.5 m and the right hand side tunnel tube of 500.9 m.

Tunnel tubes are spaced at a minimum distance of 35.0 m. The eastern tunnel portal is located at 392.3 m above the sea level, and the western tunnel portal is at 400.2 m above the sea level, resulting in difference of 7.9 m.

Grade in the longitudinal direction is approximately 1.5%. Each of the four portal structures is 12 m long.

Tunnel tube accommodates two-lane roadway (3.75 m + 3.60 m) with the overall width of 7.35 m. Transversal slope of the roadway varies from 2.5% to 3.5%.

Clear cross-sectional area of the tunnel is 56.17 m^2, but the excavation area is in the range of 71.86 m^2 to 86.26 m.

Overburden is approximately 50 m. Total of 76 400 m^3 of material was excavated.

The minimum thickness of the concrete lining is 30 cm. It is constructed of C 25/30 concrete class and total of 9 432 m^3 of concrete was used.

Construction of the tunnel has started in August 2004.

Figure 4. Strazina tunnel.

The entire section between Dugopolje and Bisko of Zagreb-Split-Dubrovnik motorway is scheduled for opening to traffic in 2007.

Strazina Tunnel shown in Figure 4 is another twin-tunnel tube further to the south along the same motorway route. It is somewhat longer than the Bisko Tunnel, with 565.4 m long left-hand side tube and 587.2 m long right-hand side tunnel tube.

Construction of the tunnel started in June 2005 with the opening to traffic scheduled in 2007.

Tunnel tubes are spaced at a minimum distance of 25.0 m. Averaged overburden amounts to 50 m. Eastern and western tunnel portals are located at 282.5 m and 278.7 m above the sea level, respectively. The longitudinal grade in the tunnel is 0.6%.

The roadway in the tunnel is 7.70 m wide, providing for two traffic lanes of 3.85 m. Transversal slope of the roadway is 2.5%.

Cross-sectional area of the tunnel is the same as that of the Bisko tunnel (56.17 m^2), while the excavation are between 70.60 m^2 and 84.05 m^2. Total of 85 800 m^3 material was excavated for this tunnel, and the amount of concrete needed for the construction amounts to 11 800 m^3.

Each of the four portal structures is 12.0 m long, and there is a cut & cover section in the left-hand side tunnel tube in the length of 24.0 m. Secondary tunnel lining is made of concrete class C 25/30, and is 30 cm thick.

2.3 *Sv. Rok Tunnel*

At the time of its construction the Sv. Rok Tunnel was the longest tunnel ever to be constructed in Croatia. It cuts through the Velebit Mountain (Fig. 5), in a total length of 5680 m.

Prior to the completion of the Sv. Rok Tunnel, the longest tunnel in Croatia was Ucka Tunnel near Rijeka, with a total length of 5062 m, which was opened to

Figure 5. Sv. Rok tunnel through Velebit mountain.

Figure 6. Ucka tunnel dating back to 1981.

service back in 1981 (Fig. 6). But, while the Ucka Tunnel is a single-tube tunnel, two separate tunnel tubes were excavated for the Sv. Rok Tunnel.

Sv. Rok Tunnel is located on Zagreb to Split motorway, on section between Sv. Rok Interchange and Maslenica.

The preparation work for the construction including clearing the site of landmines started back in 1996, but the excavation itself began in 1997 and was completed in 1999. The tunnel was opened to traffic in 2003.

Two tunnel tubes are spaced at a distance of approximately 35 m.

At the northern entrance located at 561 m above the sea level the slope of the grade line is 0.40%, and at the southern portal located at 511 m above the sea level the slope is 1.5%.

The clear cross-sectional area of a single tube of the Sv. Rok Tunnel is 58.90 m^2. The area of excavation ranges from 70.90 m^2 to 82.30 m^2.

Figure 7. Inside of the Sv. Rok Tunnel.

Figure 8. Twin-tube Mala Kapela tunnel; currently only one tube is opened for two-way traffic.

Table 2. General data on Sv. Rok tunnel.

	Unit	Value
Northern portal	(m above MSL)	561
Southern portal	(m above MSL)	511
Upward slope beginning of tunnel	(%)	1.5
Downward slope end of tunnel	(%)	0.4
Cross-sectional area	(m^2)	58/tube

The roadway in the tunnel is 7.7 m wide (2×3.85 m; refer to Figure 7) with a slope in transverse direction varying from 1.5% to 2.5%.

Tunnel tubes were excavated using New Austrian Tunnelling Method (NATM), which is the only excavation technique that can be used in the region where tunnel passes through all types of soil.

The excavation of twin-tube tunnel was carried out in two stages.

The first stage included excavation of the entire right-hand side tube and 2700 m of the left-hand side tube (almost 50% of its length).

The lining of the right tube is already executed when the second stage of excavation on the remaining portion of the left tube of the Sv. Rok Tunnel takes place. Blasting induces energy in a form of movement and impact on surrounding soil. The distance between two tubes is not large enough to ignore the effect of blasting.

Thus, the design of the concrete lining of the right-hand side tube had to take into account an additional loading due to blasting in the adjacent tunnel tube (Tkalcic et al. 2002).

Finite element analysis was used for this purpose. Data on velocity of the impact were obtained by on-site testing. Movement of the rock mass was measured in the right-hand side tube were the lining had not yet been constructed. The results provided relation between rock movement speed and time.

The main objective of this investigation was to determine excavation step length at which the existing lining in the right-hand tube can remain without reinforcement. It was concluded that from the economical point of view this goal cannot be achieved.

It would be neccesary to make blasting steps so small that it would be more reasonable economicaly to put some reinforcement in the lining.

The second intention was to keep lining reinforcement at the minimum level which would comply with the requirements of Croatian standards.

Since more than 3000 m of the left-hand tube were not excavated at the first stage, placing reinforcement in the existing lining at that length represents a substantial increase in tunnel cost.

This was the main reason that reinforcement in existing tunnel lining was to be kept at the minimum level.

Only the right-hand side tube of the Sv. Rok Tunnel is opened to traffic, while the other one serves as a service tunnel and escape route, refer to Figure 8.

Completion and opening to traffic of the second tube is anticipated when the traffic intensity requires it.

This tunnel tube contains 4 turning points, 12 stopping lay-bys and 14 cross-passages for pedestrian use.

2.4 *Mala Kapela Tunnel*

Mala Kapela Tunnel is located on Zagreb to Split motorway, on section III between Bosiljevo interchange and Sv. Rok.

The tunnel runs through both Mala Kapela and Velika Kapela massif.

It was opened to traffic in June 2005, together with the final 33 km long motorway section between Vrpolje and Pirovac, thus completing motorway connection between the two largest Croatian cities.

Table 3. General data on Mala Kapela Tunnel.

	Unit	Value
Northern portal	(m above MSL)	562
Southern portal	(m above MSL)	573
Upward slope beginning of tunnel	(%)	1.5
Downward slope end of tunnel	(%)	1.2
Cross-sectional area	(m^2)	56/tube

The initial design of the Mala Kapela Tunnel envisaged right-hand side tunnel tube of 5680 m in length, and left-hand side tunnel tube of 5683.2 m in length.

According to these, Mala Kapela Tunnel would have surpassed Sv. Rok Tunnel as then the longest tunnel structure in Croatia by approximately 10 m.

The length of the tunnel was later changed to facilitate the construction of the adjacent Jezerane viaduct. Thus, the completed structure consists of the 5760-m long right tube, and the 5761.7-m long left tube.

Two-tube tunnel was designed, with one-way traffic in both tubes. From the point of view of safety, a twin-tube system with two separate tunnels has a clear advantage over a single tube tunnel. In a case of fire or some other accident, the other tube serves as an escape route.

Longitudinal ventilation of the tunnel also contributes to fire safety in the tunnel as in the case of fire smoke is driven in the direction of travel.

Additionally, one-way traffic in tube provides for traffic safety, as frontal vehicle collisions are excluded.

The clear cross-sectional area of a Mala Kapela Tunnel tube is 56.17 m^2, thus fulfilling all the requirements of both Croatian and Austrian safety standards.

Austrian regulations and recommendations concerning safety in road tunnels were used to supplement the Croatian standards, which date back to 1973. This was only natural, as the tunnel was excavated using New Austrian Tunnelling Method (NATM).

The tunnel is designed for the speed of 100 km/h. Two parallel tubes of the Mala Kapela Tunnel are spaced at a distance of 25 m.

Tunnel grade at the northern entrance (Josipdol) is 0.82%, and at the southern portal (Jezerane) is 1.466%.

The northern portal is located at 562 m above the sea level, while the south tunnel portal is located at 573 m above the sea level, resulting in the difference of 11 meters. The highest point of the tunnel is located at 604 m above the sea level.

The roadway in the tunnel is 7.7 m wide consisting of two 3.5-m wide traffic lanes with 0.35 m wide edge strips on both sides. 90 cm wide sidewalks are elevated 15 cm, and accommodate utility ducts.

Figure 9. Twin-tube Mala Kapela tunnel; currently only one tube is opened for two-way traffic.

Figure 10. Inside of the Mala Kapela Tunnel.

The slope of the roadway in transverse direction is 2.5% directed towards the outer side of the tunnel tube.

Closed drainage system collects the water from the roadway and transfers it to a separate treatment facility.

Construction works started in 2002, and were successfully completed with excavation of both tunnel tubes.

The excavation rate depended naturally on the geology, but on occasions it reached 50 m a day i.e. 200 m a week.

Both tunnel tubes were excavated simultaneously, but at the moment only the right-hand side tunnel tube is opened to two-way traffic, as shown in Figures 9-10, while the other one serves as a service tunnel and escape route.

Completion and opening to traffic of the second tube is anticipated when the traffic intensity requires it. Tunnel tube serving the traffic has 12 emergency and stopping lay-bys.

Additionally, 6 turning points for traffic redirection (spaced at 840 m) were constructed and equipped with sliding fireproof doors providing cross-passage to the

Figure 11. Portal structure on the southern entrance to the Mala Kapela Tunnel.

second tube. There are a total of 14 inches provided in between these cross-passages, spaced at 280 m.

The tunnel is equipped with highly sophisticated traffic surveillance systems which provide 24-hour-a-day surveillance by experts at road maintenance and traffic control centres.

The minimum thickness of the concrete lining is 30 cm. It is constructed of C 25/30 concrete class.

Generally, there is no reinforcement in the concrete lining, except at the location of inches and cross-passages, sections where the large ventilations fans are installed, as well as in the first and last tunnel sections.

Waterproofing layer is installed between the primary and secondary lining.

The primary lining of each tunnel tube contains 19 000 m^3 of shotcrete, while the tunnel tube itself comprises approximately 55 000 m^3 of concrete.

Figure 12 illustrates the construction of the Mala Kapela Tunnel.

3 CONCLUSIONS

Over the last 15 years Croatia made large effort and investments in order to construct motorway network which would integrate the most distant parts of the country and provide for quick, easy and comfortable transportation of people, goods and services.

Several sections of new motorways cut through mountainous regions, thus construction workers were challenged to excavate a substantial number of tunnels, under very tight deadlines and unusually dangerous circumstances posed by landmines left over from the recent War for independence.

Many extraordinary structures have been constructed using the most modern technologies, and the record of the longest tunnel ever built in Croatia was surpassed on two tunnels.

Figure 12. Construction of the Mala Kapela Tunnel.

Most of the tunnels on new Croatian motorways are built as twin-tube structures. The vast majority of those tunnels which were initially constructed as a single-tube structure, are currently being added a second one. Two longest tunnels in Croatia – the Mala Kapela and Sv. Rok tunnels have both been constructed as twin-tube tunnels, but for the time being only one tube is serving two-way traffic as the present traffic volume does not justify the huge cost of equipping the second tube.

In addition to structural safety, much attention and resources were assigned to ensuring traffic and fire safety in tunnel structures. This was confirmed by European Tunnel Assessment Programme which put one of Croatian tunnels – Plasina tunnel on Zagreb to Split motorway – on very high second place while testing tunnels all across Europe with respect to tunnel system, lighting and power supply, communication, escape and rescue routes, fire protection, ventilation and emergency management.

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Underground Space – the 4th Dimension of Metropolises – Barták, Hrdina & Romancov (eds)

The Dulles Corridor Metrorail Project – Tunneling aspects of the Metrorail extension to Washington, DC Dulles International Airport Phase I and Phase II

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ABSTRACT: The Virginia Department of Rail and Public Transportation (DRPT) is currently undertaking the extension of Washington Metropolitan Area Transit Authority's Metrorail service to Washington Dulles International Airport. The 37 kilometers long Metrorail alignment is scheduled for completion in 2015. The project features two single-track, 700 meter long NATM tunnels at shallow depths in soft ground. At Dulles Airport two 3.3 kilometer long, single-track tunnels will be bored by TBM in siltstone rock. A 25-meter deep station will be constructed using NATM techniques. The total cost of the Metrorail project is estimated to be approximately US $4.0 billion (2006 dollars). Upon completion of preliminary engineering in early 2007 the design build contractor, a joint venture of Bechtel and Washington Group International and referred to as Dulles Transit Partners (DTP) will submit a negotiated, firm, fixed price to DRPT and the project will be implemented under a public private partnership agreement.

1 INTRODUCTION

The Dulles Metrorail project will extend WMATA's rail services from the Metrorail Orange Line in Fairfax County, Virginia to Route 772 near Ashburn in eastern Loudoun County, Virginia. This corridor encompasses several activity centers including Tysons Corner, Reston, Herndon, and International Airport Dulles (IAD) as well as emerging activity centers in eastern Loudoun County. The project alignment within the Dulles Corridor is displayed in Figure 1.

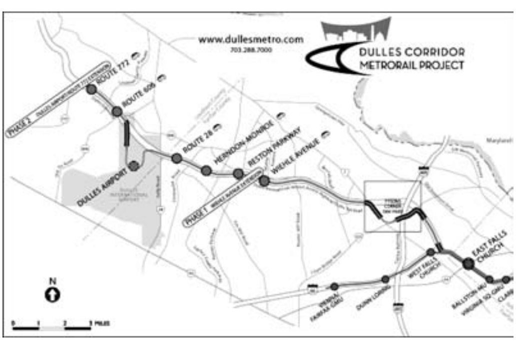

Figure 1. Dulles Corridor Metrorail project.

Rapid Transit for the Dulles Corridor was initially explored in the 1950's as part of the planning process for Dulles Airport. At that time it was decided to reserve the median of the Dulles Airport Access Highway for future transit access to the airport. Preservation of this median allows the alignment to be at grade for most of its length within the corridor. Since the initial planning the need for transit in the Dulles Corridor had been studied and although rail transit in the corridor was not part of WMATA's originally adopted rapid transit system, rapid transit service for the corridor remained a local and regional goal (Schrag, 2006).

The strong growth of the activity centers within the corridor in particular during the1990's and recent 2000's that continues today has led to inception of Metrorail in the Dulles Corridor. Current and projected, future regional growth data exemplify the need for rapid transit and its timely implementation (Dulles Transit Partners, 2006):

– Tysons Corner is the largest employment center in Virginia with 115,000 jobs and close to 4 million square meters of commercial space.
– Reston/Herndon is home of 70,000 jobs and 2.7 million square meters of commercial space.
– In Fairfax County employment is expected to increase 63 percent in the next 20 years.

– Loudoun County grew by 49 percent in the last 5 years and is currently the fastest growing county in the US.
– In the last nine years traffic on the Toll Road in Loudoun County has increased from 50,000 to 90,000 cars per day.
– Dulles International Airport employs more than 18,000 people and serves 23 million passengers per year and presently is being expanded and modernized. Modernization includes a new underground automated people mover system with multiple stations at main and mid terminals.

Regional growth and progress result however in urban and social challenges:

– Washington, DC region has the 3rd worst congestion in the US.
– The annual delay amounts to 69 hours per traveler resulting in a "congestion cost" of US $2.5 billion per year.
– 5 of 8 main roads in the corridor will be gridlocked by 2010.

The much-needed implementation of the project began with Preliminary Engineering in 2004 under a public private partnership agreement between DRPT and DTP. Other funding partners in financing the preliminary engineering effort are the Federal Transit Administration (FTA), the Metropolitan Washington Airports Authority (MWAA), WMATA, County of Fairfax, Loudoun County, and the towns of Reston and Herndon.

This extension to be known as the Silverline once completed will significantly extend the existing Metrorail system. The original system as conceptualized in the 1960's included 103 miles (166 kilometers) and was designed and built between 1969 and 2001. Additions including the Largo Line were accomplished between 2001 and 2004 extending the total system length to about 171 kilometers. The planned extension to the Airport will therefore constitute an addition of some 23% in length.

2 WMATA METRORAIL SYSTEM – COMPONENTS AND TUNNELING EXPERIENCE

A summary of the existing WMATA Metrorail system components is provided in Table 1 followed by a summary of WMATA's tunneling experience of the three decades between the early 1970's through late 1990's.

WMATA's over 80 Kilometers of subway construction provides many examples of tunneling methods and types of tunnel construction and displays a continuous development of tunnel design and construction methodology spanning some 30 years.

Table 1. Current Metrorail system.

Systemwide	Double track length (Km)	Stations (Number)
Subway Includes cut-and-cover construction	80.55	47
Surface	70.41	32
Aerial	14.84	7
Metro System (Total in 2001)	**165.79**	84
Without Largo segment		
By Jurisdiction		
District of Columbia	61.64	40
Maryland	61.55	24
Virginia	47.43	20
Total Metro System	**170.62**	86
With Largo segment added in 2004		

In the 1970's WMATA had employed tunneling methods nowadays considered an "old-standard." In soft ground methods involved mandatory dewatering for tunneling with open face digger shields, breasting and temporary support by steel ribs and lagging. These soft ground tunnels were designed for loading conditions assuming a load equivalent to full overburden. Consequently, the final tunnel lining was a rigid, heavily reinforced cast-in-place concrete structure with PVC waterstops in contraction joints as the only means of positive waterproofing. Such construction was used on the Inner City A-Redline, D-Orangeline, and Outer G-Blueline. During that time there are examples of utilizing cast iron bolted segmental lining with lead waterproofed joints between the liner segments. Cast iron linings were used for the Potomac River tunnel crossing on the Orangeline and the Waterfront Tunnel on the F-Greenline. Immersed ("sunken") tube construction was used across the Washington Channel (L-Yellowline).

For tunneling in rock drill-and-blast methods were used for excavation with steel ribs and cribbing as temporary support followed by cast-in-place reinforced concrete for final tunnel support. During this period WMATA already used a modern, gripper-type rock TBM when good bedrock conditions were present, with cast-in-place reinforced concrete lining as final tunnel support. An example is a section on the A-Redline. For the construction of large, approximately 20 meters wide mined station vaults pilot tunnels followed by multiple drift mining were employed. These openings were supported by heavy rock bolting and massive steel ribs embedded in shotcrete for both temporary and permanent support. The final structure was established as an independent architectural precast concrete structure within the mined vault. For the design of the permanent support in rock some arching effect was considered. Tunnel construction on the A-Redline under Connecticut and Wisconsin Avenues is an example for such rock tunneling.

In the 1980's soft ground tunneling was accomplished using sophisticated Earth Pressure Balance Machines (EPBM) and a single pass segmental, precast concrete lining with gaskets fabricated with tight tolerances. The tunneling was performed under the Anacostia River in adverse ground conditions with 3 bar hydrostatic pressure. It resulted in a very successful waterproofing largely as a result of well-designed and tight tolerances that were required for segment construction and gasket fabrication. This EPBM tunneling was used on two different sections under M Street namely Sections F3a and F3c on the Greenline. Successful installation of bolted segments depended on contact grouting within the time specified. On other sections an open face TBM was utilized. On a section with a low hydrostatic pressure compressed air was employed to control ground water. On another section with an open face TBM systematic dewatering was performed. Both open face TBM drives utilized a one-pass segmental, gasketed, pre-cast concrete lining which was successfully installed.

Also, in the 1980's WMATA allowed new, at that time progressive tunneling and waterproofing approaches. Consequently, in 1984 WMATA accepted the use of NATM rock tunneling proposed by the contractor. This was the first application of a dual lining NATM with PVC waterproofing in the US. It was utilized for running tunnel and station construction on the B-Redline to Wheaton, MD. The design considered arching of the surrounding ground and interaction between ground and the initial lining. Un-reinforced, thin, cast-in-place concrete lining was used for final support. Tunnel and station waterproofing was by an "umbrella type" PVC membrane with fully immersed sidewall drains on both sides of the vault. This resulted in completely dry tunnels in contrast to the A-Redline tunnels experiencing persistent leaks. At the end of the 1980's and at the beginning of the 1990's NATM tunneling was used again, but this time in soft ground for running tunnels and complicated, split station vault construction. The station was built using five different drifts. The first center drift was excavated for installation of a column line located in the middle of the station platform. Both, station and running tunnels were fully encased by a PVC membrane (Fort-Totten Station on the E-Greenline).

In the mid 1990's the NATM was used again for soft ground tunneling by employing dewatering from inside the tunnel and a grouted pipe arch as a crown pre-support to control surface settlement. The grouted pipes were installed by "directional drilling" methods under the Rock Creek Cemetery from a shaft at New Hampshire Avenue. This section was part of the Mid-City E-Greenline.

Also in the 1990's WMATA adopted a "two-pass" lining system for the circular soft ground tunnels excavated by the open face digger shield method (Outer E-Greenline tunnels; Sections E6e and E8a). Besides the need for dewatering this method also required the use of ground modification techniques such as chemical grouting systematically applied from the surface prior to tunneling (Mid-City E-Greenline tunnels; the Under/Over tunnels at Park Road & 14th Street Tunnels). The two-pass tunnel in soft ground with an initial pre-cast concrete liner usually considered as "throw-away" temporary lining was accounted for in the design of the final lining support system. The premise for this assumption was that solid, closed concrete rings were used for the initial support by not allowing any wooden wedges between segments. Rather, the pre-cast lining was required to be fully stabilized before the final concrete lining was cast. The combined liners for final support were designed considering flexibility of the initial lining and soil-structure interaction for "Short Term Loading" and all WMATA loading combinations including full hydrostatic pressure acting on the final lining for "Long Term Loading." Using these assumptions the initial pre-cast and the final cast-in-place linings share the loading combination. This allowed the use of an un-reinforced, cast-in-place final concrete lining. For the initial liner segments installed as expanded rings, success depended upon chemical pre-grouting, dewatering, immediate expansion by jacking the segments against the ground.

Depending on the nature of the soils, ground water level and difficulty in dewatering, such as from aquifers of artesian nature, it was necessary to use EPBM technology again but with the initial liners of non-expansion type but bolted segments similar to those in single-pass installations but with temporary gaskets. This installation was followed by an un-reinforced cast-in-place concrete lining and referred to as "Modified Two-Pass" with a PVC waterproofing membrane between initial and final linings. Such systems were used on the Outer F-Greenline, Sections F6a and F6c, Suitland Parkway to Branch Avenue. Here, the two-pass lining system was used for the first time with the EPBM tunneling method on the WMATA system. In this application the usual rings of four (4) reinforced concrete segments with added key segment forming rings are lightly bolted in the longitudinal joints. The gaskets in joints and the initial liner are designed for temporary hydrostatic pressure as the final waterproofing is achieved by the PVC membrane installed around the entire lining circumference. This system obviously is more costly, but it was necessary to overcome the most adverse ground and water condition where dewatering was not allowed due to environmental concerns. For the initial liner the segments were installed as bolted rings, and success depended upon water control, proper erection scheme and accomplishing contact grouting immediately behind a sealed tail of the TBM shield (Rudolf, 1997).

3 DULLES CORRIDOR METRORAIL PROJECT DESCRIPTION

The project description concentrates on the tunneling aspects of the work at Tysons Corner (Phase I) and at Dulles Airport (Phase II). The preliminary engineering of Phase I essentially followed the general plans of the Locally Preferred Alternative (LPA) selected by WMATA out of many alternate alignments studied including a tunnel at Tysons Corner. The LPA as portrayed in the approved Final Environmental Impact Statement (FEIS) is designed mainly as an aerial guideway with short tunnels through Tysons Corner.

Late in the preliminary engineering of Phase I WMATA in conjunction with a Spanish contractor and an Austrian design group strongly supported by a local developer proposed an all-underground option for the roughly 6.0 kilometers long segment at Tysons Corner. The envisioned tunnel would have been a large bore, 12 m-diameter TBM driven tunnel to accommodate two over/under tracks and stacked station platforms. It was based on a deep tunneling experience gained at the Barcelona Light Rail system recently constructed (Della Valle, 2002 and 2005). Despite strong support of an underground option by all parties involved, its realization was found to cost from US $250 to $800 million more, based on various estimates, than the mostly elevated and partially at-grade alignment. Furthermore the tunnel option would have significantly deviated from the NEPA selected and approved alignment as portrayed in the preliminary engineering documents. This new tunnel concept would have involved another environmental approval process, and additional geotechnical studies to be followed by a new preliminary engineering. This in turn would have resulted in a project delay of some 2 1/2 to 3 years. The additional projected cost for the tunnel alternative would have practically led to the loss of funding by the Federal Transit Administration (FTA). Due to these factors and the fact that delaying the relief of every-day traffic congestion in Tysons Corner by another up to 3 years, the option was found unacceptable and therefore not pursued further.

3.1 *Phase I tunneling*

The mined tunnel segment includes twin single track NATM tunnels at a length of 700 meters each and an emergency cross-passage. Short cut-and-cover sections will be utilized at the portals. These tunnels will be constructed in soft ground and will be located adjacent to existing structures and utilities that are sensitive to ground movements.

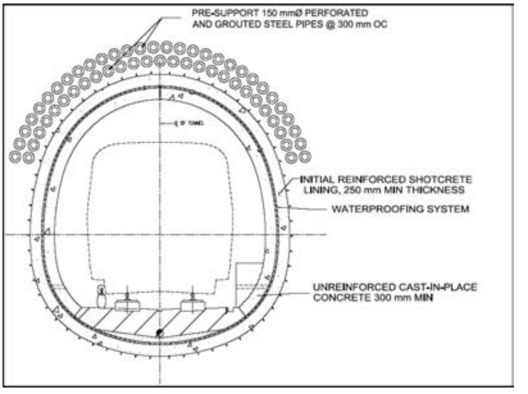

Figure 2. NATM tunnel with pipe arch pre-support.

The soils include mainly residual soils and soil like, completely decomposed rock. The residual soils encountered along the alignment are the result of in-place weathering of the underlying bedrock and are typically fine sandy silts and clays, and silty fine sands. Ground water at portal locations is generally at invert elevation, in mid-point of the tunnel alignment it rises up to the tunnel spring line.

Prominent building and infrastructure elements located in the tunnel's vicinity include an underground parking garage at a distance of some 8 meters from the outbound tunnel wall and bridge piers of the Route 123/Route 7 overpass, at a clear distance of approximately 15 meters from the inbound tunnel, as well as International Drive, a six-lane divided highway located about 4.5 meters above the future tunnel crowns. Deepest overburden cover exists at about mid-point of the alignment with nearly 12 meters. At the west portal and in the center of Route 7 the overburden cover is just 4 meters.

Because of the shallow depth, the prevailing soft ground conditions, the relatively short tunnel length, and the need to control settlements the NATM has been chosen as the preferred tunneling method. To enhance stand-up time of the soils and minimize settlements a single row of a grouted pipe arch umbrella will be utilized for the entire length of the tunnels. This will be sufficient for pre-support where the overburden is greater and surface structures are less sensitive. An additional row of pipe arch umbrellas, using closely spaced 150 mm diameter sleeved steel pipes (tube-a-manchette) will be used on the first 100 m length at the portals where tunneling is shallow with less overburden under International Drive. Figure 2 displays the double row pipe arch umbrella above a typical single track NATM tunnel with shotcrete initial lining, closed PVC membrane waterproofing system and a cast-in-place concrete final lining.

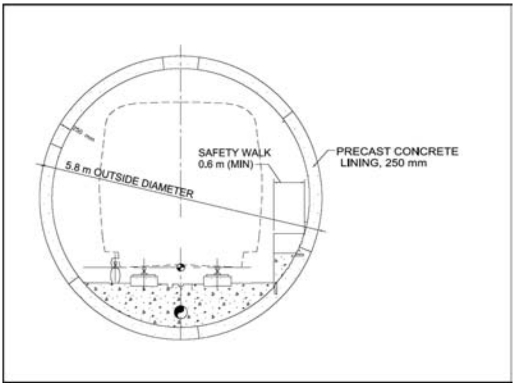

Figure 3. Typical TBM tunnel section.

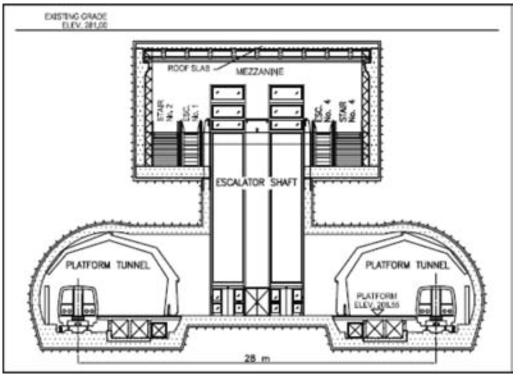

Figure 4. Station typical structural cross section.

Figure 5. Station tunnel rendering (by diDo menico + Partners, Architectural Design Consultant).

3.2 *Phase II tunneling*

The underground segment of Phase II lies within Dulles International Airport property with the metro station referred to as Dulles Airport Station in front of the main terminal. The main terminal has considerable traffic and existing infrastructure with much of the project area having a high concentration of existing utilities. The underground structures include twin single-track TBM tunnels, emergency cross passages, shafts and mined caverns for the Dulles Airport underground station. These structures will be constructed in bedrock and will be located below existing structures and utilities that are sensitive to ground movements. The host geologic formation for tunneling will be generally competent siltstone bedrock whereas the over burden includes fill, residual soils, and decomposed rock.

The TBM tunnels have an approximately 6 meter outside diameter and are about 3.31 kilometers long each. The tunnels will be constructed by either a shielded rock TBM using a single pass, pre-cast concrete, gasketed lining or a rock gripper type TBM with an initial rock support followed by installation of a PVC membrane waterproofing and a final cast-in-place concrete lining. Figure 3 displays a typical, single pass lining cross section for the TBM tunneling.

The mined portions of Dulles Airport Station will be constructed using NATM techniques in sedimentary, typically siltstone bedrock. Excavation will be by road headers. Initial support will consist of rock reinforcement and shotcrete lining. All mined station and associated structures will be waterproofed using an open, “umbrella type” waterproofing system with sidewall drain pipes. Figure 4 displays a typical station tunnel configuration at the central cross passage with 5.2 meters wide platforms. The station platform is about 25 meters below the ground surface.

To allow for a twin station tunnel configuration, where there are two parallel station vaults, the centerline track-to-track distance is 28 meters. Both station platform tunnels are 183 meters (600 feet) long and unobstructed by vertical circulation. The station platforms are connected with cross-passages between the station tunnels. Access to the platforms is provided by a central access structure located between the two station vaults. Architectural rendering for the station tunnel configuration is shown in Figure 5.

All station construction will be mined except for the mezzanine and ancillary rooms, which will be constructed using cut-and-cover techniques. Mined station construction has been selected to minimize disruption to airport activities. Surface disruptions will therefore generally be limited to Mezzanine and ancillary room construction using cut-and-cover excavation while maintaining airport pedestrian circulation above, except for the time period when the mezzanine box will be connected to an existing pedestrian tunnel “Node” that will provide Metrorail Station access.

4 IMPLEMENTATION

4.1 *Public Private Partnership (PPP)*

The project is being implemented in a Public-Private-Partnership under the Public Private Transportation Act (PPTA) an innovative project delivery framework as established by the Virginia Department of Transportation (VDOT) in 1995. Its implementation is in accordance with the guidelines as amended by the General Assembly in 2005 (The Commonwealth of Virginia, 2005). The essential goals of the PPTA are to encourage investment in the Commonwealth by creating a more stable investment climate and increasing transparency and public involvement in the procurement process. According to the guidelines the private entity charged with project implementation is required to provide certain commitments or guarantees and enters into a mandatory risk sharing.

4.2 *Design and construction*

The project is being realized under a design-build contract. The design-builder, Dulles Transit Partners (DTP) is required to initially develop preliminary engineering for the rail project. The cost for the preliminary engineering is shared between the design-builder and the project partners, DRPT, FTA, MWAA and the counties of Fairfax and Loudoun. The preliminary engineering then forms the basis to develop a fixed firm price by the design-builder. To maintain previously established budget limits this results in design challenges and the need to optimize design and construction methods to build to budget. Consequently, many design iterations are required during preliminary engineering. The design and construction team constantly weighs the benefits of underground space to keep everyday routines undisrupted versus its increased cost when compared to at grade and above ground construction.

Value Planning (VP) and Value Engineering (VE) exercises are a central activity of the design development in pursuit of the most economical approach with least impact on the surroundings. In Phase I these exercises led to a series of transformations of the underground segment at Tysons Corner. This alignment was initially envisioned as deep, 1.6-kilometer long twin TBM tunnels and a roughly 24 meter deep underground station constructed by cut-and-cover methods within Route 7, a busy traffic artery. Analysis of construction cost however favored the implementation of the short NATM tunnels with a quasi at-grade station within the median of Route 7 at a cost saving of roughly US $200 million. In Phase II the VP exercises led to selection of a deep TBM tunneling and NATM station construction in rock instead of a cut-and-cover excavation for station and running tunnel construction originally depicted in the FEIS. Since the rock formation at the Airport is close to the surface this selection resulted in considerable cost and schedule savings. This construction will also considerably reduce impacts on the Airport operation. VE exercises, which are to follow, will search for further cost reductions; if successful these will become the basis for construction.

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Experience from architectural design and build of the underground space

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ABSTRACT: The paper presents meditations, examples and visual prompts of architectural design concerning some Prague metro stations and their incorporation into the city organism and road tunnels of the Prague road ring. There are mentioned also portals of tunnels and their connection with neighbouring bridge – two engineering works, which are applied in close co-operation of architects.

1 GENERAL THOUGHTS ABOUT ARCHITECTURAL DESIGN OF TUNNELS AND UNDERGROUND SPACES

The architectural solution of underground spaces and tunnel interiors is a special discipline, where the arguments of the static and construction needs seem to have the priority. The cooperation of statics and tunnel engineers with architects is sometimes on the edge of common interest, but still needed and contributional. Underestimating of architectonic and aesthetic qualities of such a work can be dangerous. Even a useful and well-functioning technical work can be devaluated in eyes of public this way.

As an introduction, let us spend a while with several ideas of our big theoretist of aesthetic, arts and architecture, professor Mukarovsky:

While thinking of architecture, we come to a need to define an aesthetic norm; By professor Mukarovsky has an aesthetic norm the characteristic of a rule, which is but not fixed, static, but which is changable in time. Rule with dynamic validity. For example, Fechner's so-called experimental aesthetic was based on experimentally detected and further appointed basic aesthetic

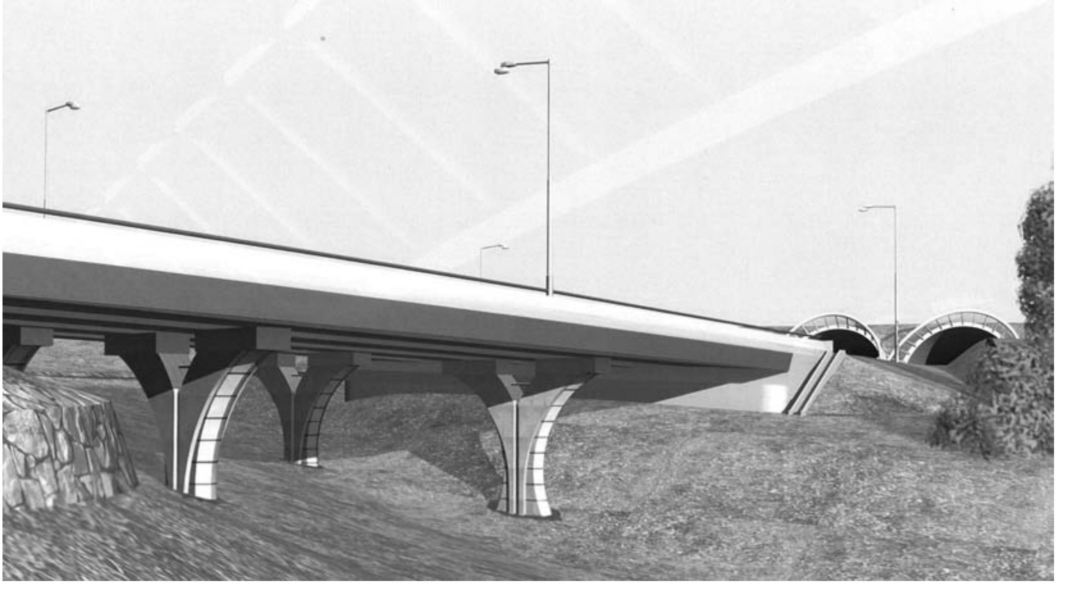

Figure 1. The cooperation of statics and tunnel engineers with architects.

norms. But their further simple applications failed to represent the ideal aesthetic sample.

My personal conviction in a best way represents Mukarovsky's quotation:

If we look at the history of art and architecture from the position of the aesthetic norm, then we see the history of revolts against rulling norms.

This introduction has an ambition to show, that rating and judging the architecture as an art discipline can be problematic and we should keep in mind, that our judgements and objections are rather of subjective and relative character.

The statement about the changebility und unpredictability of the aesthetic norms we should not forget. Therefore more than trying to appoint flatly fitting criterions, we should learn how to be tolerant and pluralist. Because tolerance and pluralism lead to richness, variability and spectacularity of the creative works around us. This means freedom of choice and so plurality as a feedback returns from our society.

Upon the positive influence of aesthetic function in arcitecture on the society agree a big amount of philosophers and theoretists from the times of Aristotel to our days. Professor Mukarovsky comments it: With the means of aesthetic value appeals the art work directly on the man's thinking process and behavior. In comparison, the science or the philosophy needs to pass through the man's willing and thinking process to be able to touch him and influence.

Therefore the aesthetic function means much more than only "the spume on the surface".

At the underground works, tunnels and engineer works, the function, safety, constructive and technical demands are widely pointed out, while the architectonic solution and aesthetic function can be considered to be "the spume on the surface". This attitude is often to be seen as an argument for the financial-costs cut. This attitude represents the narrow minded opinion, that on something, which is not quite important, such as an architecture, the expenses can be radically lowered. These pressures can be very dangerous. Because the simplyfied thinking is tempting.

This article above all refers to some underground stations, highway tunnels and their portals.

2 THE ARCHITECTONIC CONCEPTIONS AND ALTERNATIVES TO THE SOLUTION OF UNDERGROUND STATIONS

Interesting is how the architectural concept of subway stations is approached in different cities:

For example, in Stockholm's subway, the sweedish architects created so-called Cave architecture by using

Figure 2. Design of interior is binded with circle construction of the tunnel.

a local 2 miliards years old granit in a station's interior. In Marseille the mayor at the time, monsieur Gaston Defferce, enforced the idea of specifying each underground station with the local symbols. In the station Sainte-Marguerite was placed a composition reminding the watter-mills on the river Huveanne, which belonged to this fertile land in history.

In Buffalo, USA was organized exhibition, where the public voted for the most favorite art works and conceptions to be used for local subway's design.

Interesting principle was used in Vienna's: At the striked underground stations especially, the dominating idea lies in the division of platform space and railway space. By the highlighting the middle arc's edge, where it penetrates the side arc and placing the light-source on this edge, where the platform is ending, is clearly shown, which space is dedicated to the public and is ment to be pleasant and bright and which space serves as a tunnel-route for the train. That one is left in a rough condition and is rather dark. This difference factor functions as a warning: here the safety of platform is ending. The artistic principle of this extreme contrast further creates a particular tension – dynamism and let the interior talk itself. On the other side stands the conception of the Prague's metro, where even the wall behind the railway is worked out with the surface materials and is intensivelly delighted.

However contradictory these two attitudes are, both seem to have certain reason and bring the contribution to the interior design.

Mr. Liebaerse, president of the comittee for art for the Brusel's metro says:

"The people don't travel by metro for their pleasure, they have to face the stressful atmosphere, closed in a train moving through the dark tunnel. Therefore we try to lead our artists to create bright interiors, which will humanize this oppressive clima."

The fact, that these architectural works are fully under the ground and there are no fascades, which a man can perceive and overview, can be deprivating for a human beeing. This kind of architecture doesn't have an outter volume and can't join the common environment and that can be frustrating.

On the other side in the underground stations is extremelly visible the original characteristic, which differentiates architecture from the other visual-arts such as painting, sculpturing: the possibility to visit and perceive it from inside.

In any case the underground architecture works function as a vital spring for the city organism. The presence of underground traffic net improves the city functions, urbanistic development and architectural design of the buildings in a close neighbourhood.

Such architectural works bring advantages to many people, more than most of other public buildings and that's where lies their strength, competency and reliability.

3 LIGHT AND COLOUR

Very important part of architectural design is colour and ilumination of the surfaces. The undeground spaces, above all the artificial delighted ones, seem to tolerate brighter and richer colours, than the surfaces reflecting the day-light.

Rich encouraging colorcode and sufficient amount of light can ballance the oppressive atmosphere under the ground.

Using the colours in highway tunnels can provide better orientation and safety of moving traffic: In the Radotin tunnel on the Prague road ring are used two tones of blue as a basic colour together with the light gray colour of concrete. As we are approaching the emergency zones, the blue is alternated first by dark and as we come closer , by light yellow colour. This change of colour signs the existence of emergency exits or the end of the tunnel. In front of each portal, the light yellow changes instantlly in a light gray tone of concrete. Therefore the leaving the tunnel is decent.

Figure 3. Expressive colourfulness in the underground of the Hloubetin station.

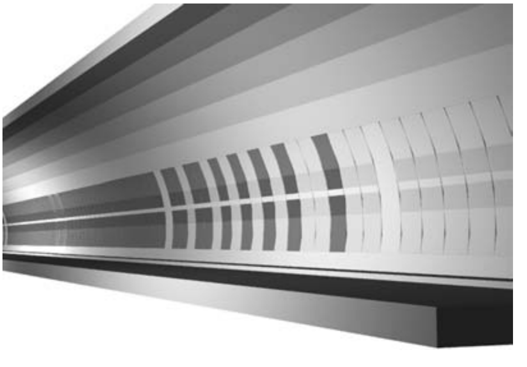

Figure 4. Coloured solution of the road tunnels Radotin.

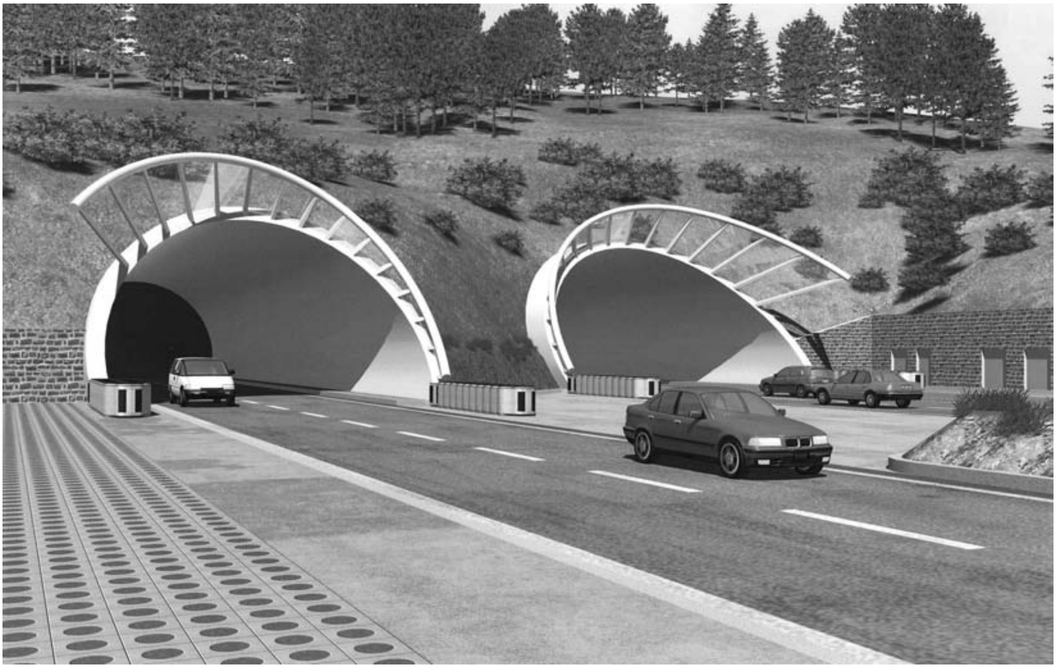

Figure 5. Tunnel portal Radotín – authoritative element.

The light yellow, already delighted surfacecould feel agressive in contrast with the outter green.

This basic colourcode is symptomatic for the whole Radotin segment of the Prague road ring, where the functions of the colours are: informative, architectonic and safety.

4 TUNNEL PORTALS

The tunnel portals sign the beginning and the end of the underground route and at the same time mark the hidden underground part and therefore should have the representative character.

The function of the portals is not to radically disturb the environment, respect the landscape and show to the approaching in advance, that they are entering the undeground route.

The tunnel portals on the Prague road ring have the appearance of a fan bowing over the cutted arch-segment of the tunnel profile. This shape is a result of an effort to create a shield protecting the entry against falling rocks and the authoritative element serving as an important orientation point for the higway travellers.

At the portal of the railway tunnel in Brezno, the main motto was to connect the surrounding rough terrain with the entry by the natural materials. Therefore the system of supporting walls made of gabions was used to shape the portal.

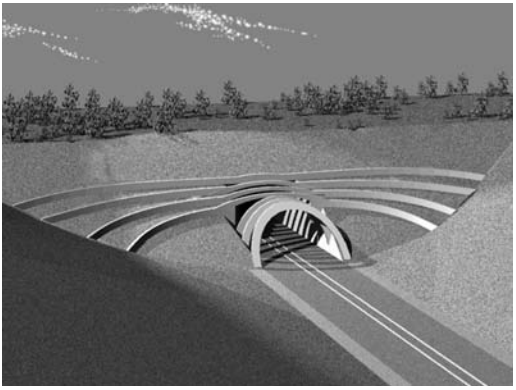

Figure 6. Railway tunnel portal Brezno.

5 THE MUTUAL INSPIRATION OF CONSTRUCTION AND ARCHITECTURAL DESIGN

The construction of tunnel works can expressivelly influence its interior appearance and also engineer works in its neighbourhood.

The west portals of the higway tunnel on the Prague road ring at Lochkov are in the close neighbourhood of the bridge over the Nightingale valley. This locality touches the protected area of natural reservation, therefore the designing process of the both works was

Figure 7. Tunnel portal Slivenec – authoritative element.

accompanished with increased attention and sensibility. For the both objects the common base idea was used and the mutual influence on each other's design is visible: The specific architecture of the fan-portals appeared also on the bridge's soft arch-shaped piers.

6 JUDGING THE ARCHITECTURAL AND ARTISTIC DESIGN OF UNDERGROUND SPACES

While we pass a judgement on the aesthetics of underground spaces (but also other architectural works), we have to admit, that all instructions how to set our rates are very problematic and relative.

Although we should stay watchful and resistant against the absolute judgements, still we can learn: Especially the cooperation of an architect and static – tunnel engineer is contributional, because the contradictory professions can enrich each other and in this cooperation the next impulse and inspiration for the creative work can be founded and cause, that the new aesthetic (or the common) norms are born. And their validity grows upon the norms which are passing over.