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Design and construction of tunnel portals under difficult geotechnical conditions

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Abstract

Between the German city of Dresden and the Czech capital Prague a 4-lane highway is under construction. At the outskirts of Dresden it will cross the narrow and deeply eroded Weißeritz Valley by a bridge. Two twin tube tunnels under oblique angles join the bridge out of the rocky valley flanks. The different geotechnical conditions of these 4 tunnel portals, their design and construction are described within this publication.

The two southern portals of the tunnel Dölzschen are situated within a vertical 65 m high former quarry slope, 25 m above the valley floor. The slope proved instable due to toppling failure and exposed serious rockfall risks. Systematic anchoring, scaling, rock bolting, netting and filling under extreme access conditions eliminated these risks. Tunnel excavation started from the slope surface without any precut or conventional portal construction.

The opposite northern portals of the tunnel Coschütz are situated in more gentle rock slopes. After removal of debris, excavation of the eastern tunnel started in a 30° slope striking oblique to the driving direction, without any precut, too. Because of the gentle slope inclination the cross section was not earlier completely covered by rock than 30 m behind.

The western tunnel started from an optimized short, narrow and steep precut. Its flanks were stabilized by reinforced shotcrete and rock bolts. Because of the small distance between the tubes rock bolts were drilled into the cross section of the neighboring tunnel.

Tunnel excavation was made by drill and blast operation, preliminary support consisted of steel arches, reinforced shotcrete and radial rock bolts. Thorough survey of the bolt’s coordinates and direction prevented them from meeting each other. Rock bolts were of the permanent type to prevent the final lining from being obliquely loaded. Systematically arrayed measuring devices checked deformations of slopes and tunnels.

Zusammenfassung

Der Neubau der Bundesautobahn A 17 wird Dresden und Prag miteinander verbinden. Die vierspurige Autobahn quert westlich von Dresden das enge und tief eingeschnittene Tal der Wilden Weißeritz mit einer Brücke. An beide Brückenwiderlager schließen mit schiefwinkligem Anschnitt doppelröhrige Tunnel an, deren Portale in den felsigen Talflanken liegen. Die unterschiedlichen geotechnischen Verhältnisse, die Überlegungen für einen jeweils optimalen Entwurf und der Bau der portalnahen bergmännischen Tunnelabschnitte sind Gegenstand dieser Veröffentlichung.

Die Südportale des Tunnels Dölzschen liegen in einer vertikalen, 65 m hohen Felswand, 25 m über dem Talgrund. Die Felswand erwies sich als global kippgefährdet, außerdem lagen flächig verteilte erhebliche Steinschlag- und Felssturzrisiken für Bau und Betrieb vor. Die Kippgefahr wurde durch ein Systemraster aus vorgespannten Felsankern, die Steinschlag- und Felssturzrisiken durch bergsteigerisches Beräumen mit dem

Brecheisen, Felsnägel und Steinschlagschutznetze eliminiert. Der Tunnelausbruch begann von einer Vorschüttung direkt in der Felswand ohne Voreinschnitt im Schutz einer kurzen Luftbogenstrecke.
Die gegenüberliegenden Nordportale des Tunnels Coschütz befinden sich in wesentlich flacheren Hängen. Nach dem Aushub einer mehrere Meter dicken Abraumhalde begann der Tunnelausbruch der östlichen Röhre asymmetrisch mit einer Luftbogenstrecke in einer spitzwinklig zur Vortriebsrichtung streichenden, 30° mit dem Vortrieb steigenden Felsfläche, ebenfalls ohne Voreinschnitt. Wegen der flachen Neigung der Böschung war der Tunnelquerschnitt erst nach 30 m vollständig überdeckt.

Für den Vortrieb der westlichen Tunnelröhre mussten erst eine zwischen den Röhren vorhandene Felsnase teilweise abgetragen, die westlich anschließende zurückspringende Felswand vertikal gestellt und gesichert und Felsaushub in der rechten Flanke getätigt werden. Der damit entstehende Voreinschnitt wurde so kurz wie möglich und ohne seitliche Arbeitsräume ausgeführt. Die Sicherung der Böschungen erfolgte mit bewehrtem Spritzbeton und Felsnägeln. Wegen des geringen Abstands der beiden Röhren wurden die unteren Lagen der radialen Ausbruchsicherung durchgeankert. Der Tunnelvortrieb begann im Schutz eines geankerten Portalkranzes und ebenfalls von einer Luftbogenstrecke aus.

Der Tunnelausbruch erfolgte im Teilausbruch als Sprengvortrieb mit einem Verbau aus Stahlbögen, bewehrtem Spritzbeton und Felsnägeln. Ein gegenseitiges Anbohren der engständigen Anker und Felsnägel wurde durch präzise Angabe von Ansatzpunkten und Richtung und sorgfältiges Einmessen vermieden. Um eine asymmetrische Belastung der Innenschale aus dem schiefwinkligen Anschnitt zu vermeiden wurde die radiale Ankerung der Tunnelleibung mit Dauerfelsnägeln ausgeführt. Das Verhalten der Böschungen und der Tunnellaibung wurde durch ein systematisches Messprogramm überwacht.

Résumé

Par la construction de l’autoroute A 17 les villes de Dresden et Prague seront reliées. Cette autoroute traverse la vallée étroite et profonde de la rivière Weißeritz par un pont, près de la ville de Dresden. Aux deux boutes du pont commencent en coup oblique deux tunnels dont les portails se trouvent dans des pentes rocheuses. Les conditions géotechniques différentes, les considérations d’obtenir des designs optimaux et la construction de ces portails selon des différents designs sont objets de cette publication.

Les portails du sud du tunnel Dölzschen sont situés 25 m sur le fond de la vallée dans une pente rocheuse verticale d’une hauteur de 65 m. La pente ne se prouvait pas suffisamment stable contre le basculement et exposait des risques sévères de chutes de pierres. La sécurité contre le basculement était produite par un système d’ancres précontraintes, le risque de chutes de pierres était éliminé par enlever les blocs instables en grimpant avec le pince- monseigneur, par des ancrages et par des filets de protection contre les chutes de pierres. Le creusement des tunnels commençait immédiatement dans la pente, sans une prétranchée courte sous la protection d’un tronçon avec cintres posés à ciel ouvert.

Les portails du nord d’en face du tunnel Coschütz sont situés dans des pentes considérablement moins inclinées. Après l’excavation d’un terril d’éboulis le creusement du tube d’est commençait dans une pente inclinée par 30 degrés, oblique à la direction de l’axe du tunnel. On avait renoncé à une prétranchée. A cause de l’inclination modeste de la surface le profil du tunnel n’était pas couvert complètement par la roche qu’après 30 m.

Pour le creusement du tube d’ouest il fallait d’excaver d’abord partiellement une barre rocheuse que se trouvait entre les deux tubes, mettre en vertical la pente voisine à la droite, au milieu du profil du tunnel et excaver le flanc droit. Toutes les surfaces d’excavation devaient être stabilisées par des ancres et du béton armé projeté. La prétranchée étant produite ainsi, était désignée et réalisée comme courte et étroite comme possible, sans des espaces disponibles pour le travail. A cause de la distance petite entre les deux tubes ils étaient joints par des ancres. Le creusement du tunnel d’ouest était fait sous la protection d’un renforcement du portail ancré et d’un tronçon court. avec cintres posés à ciel ouvert.

Le creusement des tunnels se roulait par avancement à l’explosif, le soutien avec des arcs d’acier, du béton armé projeté et des ancres d’acier forées. Le coup des ancres étant serrées à trois dimensions, était empêché par le calcul pénible des coordonnées et des directions et leur contrôle en chantier. A empêcher des poussées asymétriques par le coup oblique entre la direction des tunnels et l’orientation des pentes toutes les ancres radiales dans les tunnels, normalement temporaires, possèdent une double protection contre la corrosion. Le comportement de toutes les pentes excavées et des tunnels était surveillé par des instrumentations et mesurages systématiques.

Introduction

To fasten traffic between the Czech Republic and Germany a new highway is under construction, connecting the Czech capital Prague to the German highway system. Near Dresden the new highway crosses a narrow and deeply eroded valley. The highway comes from the north through the 600 m long Dölzschen Tunnel, crosses the valley by the 280 m long Weißeritz Bridge, 35 m above its ground, and leaves it through the 2.300 m long Coschütz Tunnel. Both tunnels are twin tubes, having a width of 16 m and a height of 12 m. Fig. 1 shows the situation.

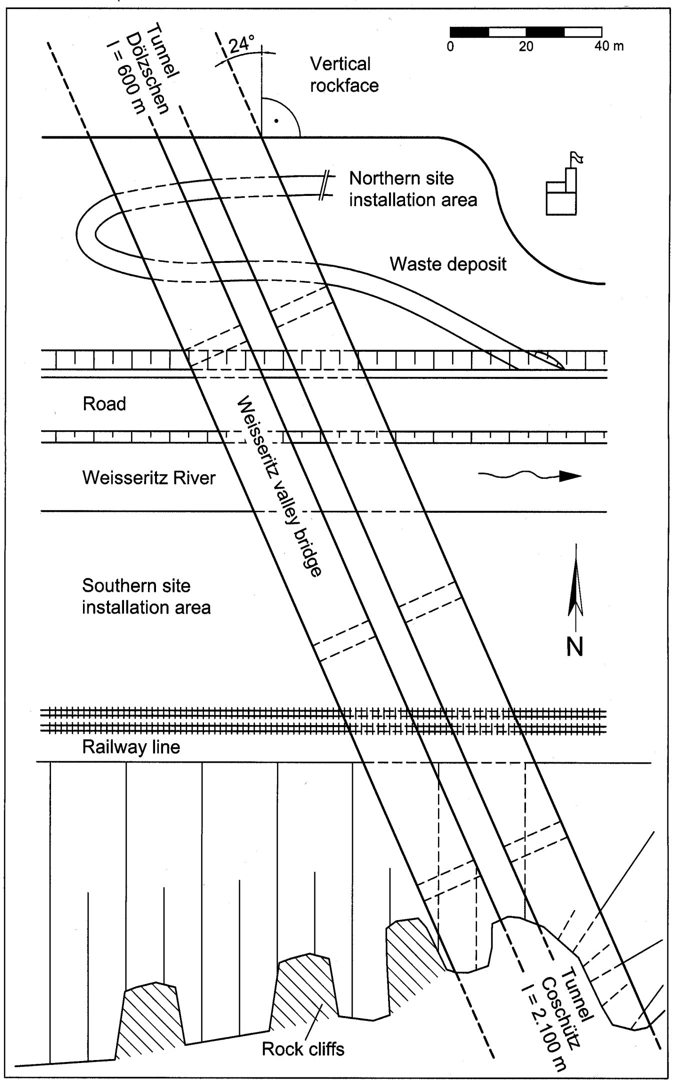

Figure 1: Crossing of the Weißeritz Valley by a bridge and the two adjacent tunnels Dölzschen and Coschütz.

The Weißeritz valley is cut into very hard Syenodiorites and is narrow, therefore. Both valley sides expose rock slopes. The right or southern one has a natural inclination between 25 and 35° with vertical cliffs up to 15 m high. Down from the crest the eastern part of the slope is covered by debris from a quarry behind the crest. The left or northern valley side was occupied by another quarry transforming the natural slope to a 65 m high vertical artificial rock slope. The portals had to be constructed within these slopes. Because of the different morphology and rock failure modes of their locations 3 different approaches had to be selected.

Design criteria:
The general design criteria were amongst others:

- Joint pattern and slope stability
- Morphology
- Optimization of cost and construction time
- Working safety.

TUNNEL PORTALS DÖLZSCHEN

Both southern portals of the tunnel Dölzschen had to be placed 25 m above the valley floor within the inaccessible 65 m high rock wall. The angle between the tunnel axis and the slope was 65°, thus loading of the lining of the portal blocks would be oblique The northern buttress of the bridge had to be integrated into this rock wall together with the portal blocks (Fig. 2).

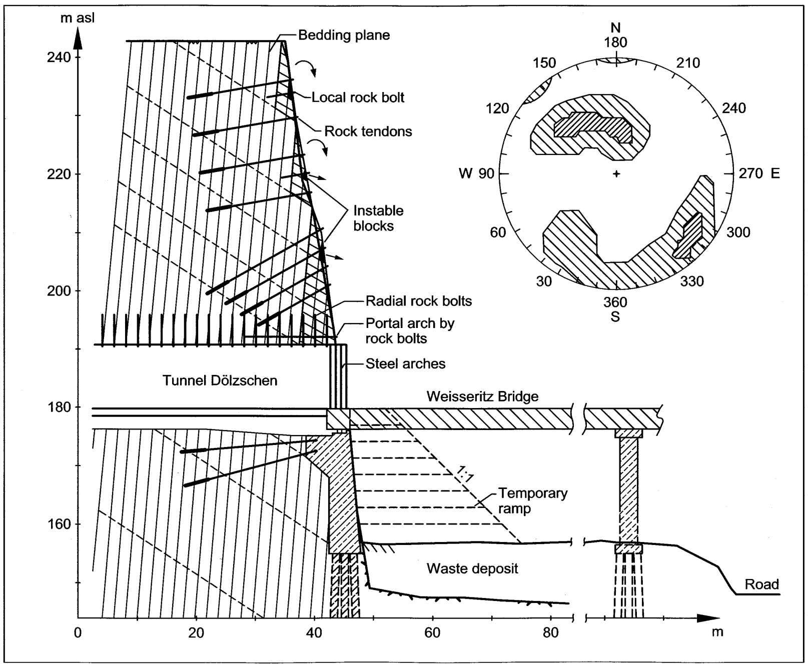

Figure 2: Southern portals of tunnel Dölzschen and northern buttress of the Weißeritz Bridge, cross section and stereographic projection of the joint pattern.

With the beginning of the final design it became obvious that the global stability of the rock wall was insufficient and that it was bound for toppling. Detailed mapping of the slope from a 65 m high mobile crane exposed a lot of instable pebbles, boulders and areas with volumes up to 100 m³ on the face leading to severe risks of rockfall and rockslides during construction as well as during operation of the later highway. A joint survey exposed open joints dipping steeply into the slope. These results led to an unforeseen, time consuming and costly rock slope stabilization program, prior to tunnel excavation.

A less costly and much faster alternative to the mitigation measures against rockfall and minor rockslides would have been an elongation of the final lining of about 15 m towards the valley. Working safety would have been achieved by previously removing the bigger blocks and preliminary netting, rolled down from the crown. Because of environmental concerns related to landscape protection such a portal structure was not allowed. No concrete or other building materials should be exposed in the rock face. From the same reasons the buttresses of the bridge had to be integrated in the tunnel portal.

Under the boundary conditions above described the following solution has been selected.

- Scaling of the slope face from the crown to the bottom by hand and crowbar;
- Stabilization of all instable rocks being to large or supporting others by rock bolts

- Covering the whole rock face above the portals by permanent steel rope rock fall nets and the lower areas by recoverable respectively preliminary ones.
- Construction of a 25 m high ramp by geotextile reinforced soil.
- Stabilization of the whole rock face by prestressed rock anchors to meet the required safety against toppling failure from mobile cranes.
- Reinforcement of the rock above the later tunnel opening by a collar of rock bolts.
- Supporting the tunnel mouth by a short array of steel arches covered by reinforced shotcrete.
- Starting excavation by drill and blast operation by .8 m deep steps.
- Stabilization of the tunnel walls by radial permanent rock bolts on the base of a stability assessment, shown by fig. 4.

Work started from the crown by mountaineering, drilling rigs and men being suspended from ropes, and finished from working platforms hanging on large mobile cranes positioned above and below the rock by drill and blast wall. The rock wall being stabilized, a 25 m high ramp was built by reinforced earth as an access and working platform for tunnel excavation. The tunnels were excavated subsequently, temporary support being provided by steel arches, reinforced shotcrete and radial rock bolts (Fig. 3). Excavation was split into crown, bench and invert. The asymmetrical loading of the first tunnel blocks by the oblique angle between the tunnel axis and the slope face was avoided by replacing the usual temporary rock bolts of the preliminary lining by permanent ones.

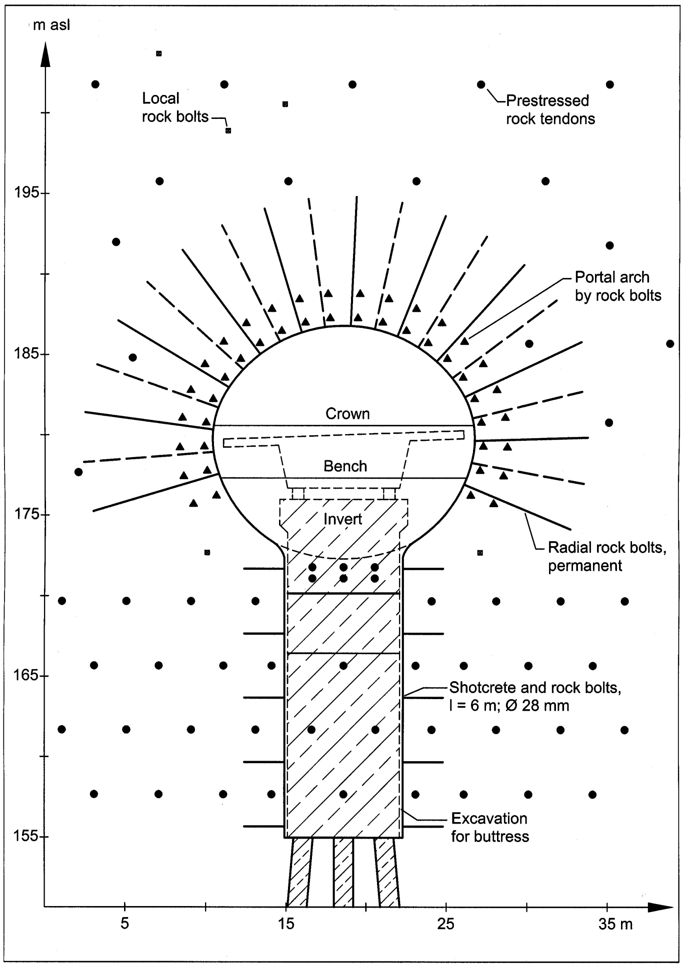

Figure 3: Tunnel Dölzschen, cross section with preliminary tunnel support and excavation for the buttress of the adjoining bridge.

On that half of the cross section of the tunnels, showing an angle from 65 to 100° between the tunnel axis and the rock slope, the necessary rock cover of the bolts required a drilling direction into the rock for the first rows of the radial rock bolts within the tunnels. This direction was turned into a direction perpendicular to the tunnel axis with advancing depth of the excavation. All rock bolts and anchors within the rock slope were drilled perpendicular to the slope surface with an inclination of 10° downwards to ease mortar injection. Because of the dense 3- dimensional grid of the different anchor systems and directions a thorough planning and control of all drillings, regarding the coordinates of their positions and their directions as well, was conducted. Thus, only one rock bolt was cut and had to be replaced.

Within 10 month only, 12.000 m² of rock face were cleared, 9.000 m² netted, 1.400 m rock bolts and 1.500 m of prestressed anchors were installed. The first 30 m of the tunnel excavation followed within 3 month, including 7.000 m³ of rock blasting and removal, and the installation of 80 t of steel arches, 1.450 t of shotcrete and 4.700 m of rock bolts.

EASTERN TUNNEL PORTAL COSCHÜTZ

At the first glance the geotechnical situation of the northern tunnel portals and the buttress of the bridge showed a lot of correspondence with the Dölzschen side. The portals and the buttress were situated 25 m above the valley, the angle between the tunnel axis and the driving direction was about 65° as well and the joint pattern was identical. But because of the inverse slope orientation the well-separated bedding joints were dipping steeply towards the valley now, inducing planar sliding failure of cut slopes (Fig. 4).

Figure 4: Northern portals of tunnel Coschütz, situation and design of tunneling operations.

For the eastern portal the strike of the slope was oblique to the driving direction of the tunnel, as had been the case on the opposite valley side, but its inclination was 30°, only. Due to this different morphological and geotechnical situation another approach for the portal design had to be selected.

The slope was covered by a 3 to 5 m thick steeply graded layer of loose barren rock from a quarry above, discharged under the natural angle of friction without any compaction. Its stabilization for a cut would have required a strong support.

In accordance with this situation the following solution was selected (Fig. 5).

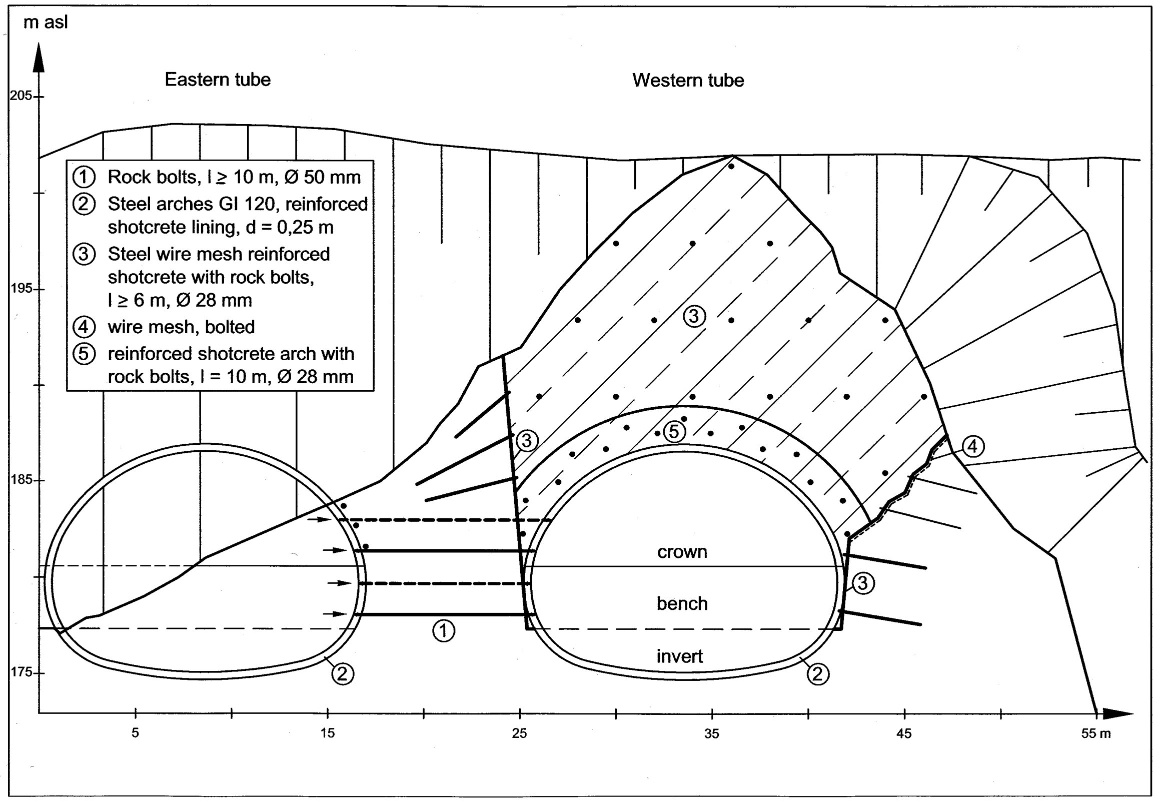

Figure 5: Tunnel Coschütz, cross section with arch shield for the eastern tube and precut for the western one.

- Removal of the debris above the foundation of the buttress of the bridge including the material above the tunnel down to the underlying rock.
- Abstention from a precut; erection of a shield of 4 free standing shotcrete covered steel arches being connected by spreader at the foot of the slope behind the buttress of the bridge.
- Starting excavation by drill and blast operation from below this shield by .8 m steps, oblique to the strike of the slope. After each step an arch was added to the shield. This procedure resulted in a low cut slope on the right side at the beginning, whereas the remaining cross section of the steel arch kept in the free air. By continued excavation the height of the cut increased, becoming a part of an undercut arch. After 25 m of excavation the arches were covered by rock completely and the usual tunnel excavation by mining operation could start.
- Stabilization of the cut surface and later the tunnel walls by reinforced shotcrete and permanent rock bolts on the base of a stability assessment, shown by fig. 4, immediately following excavation.
- Because the distance between the two tubes was not more than 5 m, it was decided to drill the bolts in the right flank into the cross section of the western tunnel and fill its angular space by mortar over the full length. During the following excavation of the western tube, the ends of these bolts would be freed, an anchor plate fixed on it and a certain pre- stress applied.

Thus no precut was necessary, no anchor collar around the crown was required. Tunneling started by the first blast. The shield of shotcrete covered steel arches provided a good outer formwork for the final lining.

WESTERN TUNNEL PORTAL COSCHÜTZ

The morphological situation of the western tunnel was considerably different from both the previously described ones. A steep rock ridge separated the two portal locations. The ridge was partly inside the cross section of the tunnel and had a nearly vertical western slope. The adjacent slope was nearly vertical, its face laying 10 m behind the front of the nose. Its strike angle was perpendicular to the tunneling direction. A rock slope with an inclination of about 60° built up the right flank of the western portal with its foot inside the cross section (Fig. 5).

All rock faces exposed rockfall risks and rock slides up to100 m³. The above described bedding joints would lead to planar sliding once undercut prohibiting vertical cuts perpendicular to the tunnel axis without anchoring. Both sides showed wedge sliding.
Under such complex morphological and geotechnical conditions a precut with a clearly defined geometry seamed the best solution, but it should be as short and narrow as possible, with almost vertical slopes and no space between the arches and the rock surface to fasten excavation and minimize subsequent backfill as well as providing a stiff horizontal embedding of the arches. It was executed in the following steps.

- Excavation of the ridge with a vertical slope to the west and no working space between the back of the tunnel lining and the slope surface; stabilization of the face by reinforced shotcrete and rock bolts. The hillside slope had to follow a vertical fault plane, striking perpendicular to the driving direction and forming the later beginning of tunneling. This plane had to be reinforced against buckling and toppling by the above described means, too.
- Excavation of the lower part of the right flank under the same rules; scaling of the higher sections outside the excavation and stabilization of remaining risks by rock bolts and fillings, finally by preliminary rock netting.
- Excavation of the back slope under the same rules. All excavations were done by drill and blast and simultaneously in vertical steps and berms as shown by fig. 5.
- Reinforcement of the rock above the later tunnel opening by a collar of rock bolts and reinforced shotcrete.
- Support of the tunnel mouth by an array of steel arches covered by reinforced shotcrete within the precut.
- Starting excavation by drill and blast operation by .8 m steps.
- Stabilization of the tunnel walls by radial permanent rock bolts on the base of a stability assessment, shown by fig. 6. To stiffen the lining a bottom arch was installed.

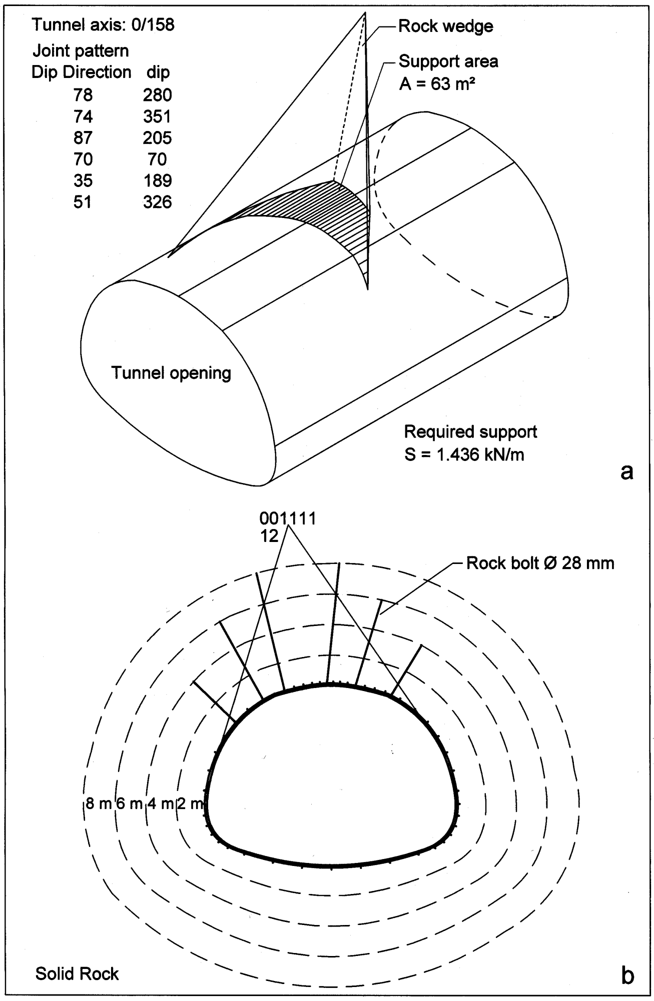

Figure 6: Analysis of failure mechanisms and loading of the tunnel lining.

All stabilization measures of rock slopes were designed on the base of joint mapping, including surveying of fault traces regarding their orientation and the resulting failure mechanisms. The mechanical model and the algorithm of toppling failure is described by SPANG & KARDEL (2002). By checking the development of joint openings to depth the instable volume could be restricted to surface near areas. The radial rock bolts were designed on the base of rigid rock bodies, limited by joints, as shown by figure 6. For the dimensioning of tunnel arches a circuit analysis program was used. The loading of the final concrete tunnel lining of the tunnels was computed on the base of loading cases derived from the above described failure mechanisms. Of course all excavations were supervised by systematic installation and control of extensometers, convergence measurements and geodetic survey of all critical reference points according to Eurocode 7.

Conclusions

All rock engineering and construction requires a sound and reliable geotechnical exploration. Too often the consequences of inadequate geotechnical investigations are time delays, cost overruns and claims. Mapping of joints in relation to existing steep rock slopes requires a direct inspection of the rock surface either from mobile cranes or other lifts or preferably by mountaineering. Inspections from the foot of the slope will never reveal the real state of a rock mass, because of the lack of the third dimension. Because there is no second chance the attached personal must have the experience to assess the local situation as for their stability, risk potential and adequate stabilization immediately during the inspection. Global instability can be recognized in the field mostly and verified later in the office by the powerful routines described by JOHN (1971) and others. Global instabilities detected in the office frequently proof unrealistic, for example, because of self-locking of toppling strata or other mechanisms.

Flexible design of rock structures can save a considerable amount of construction time and money, as was the case with the described tunnel portals. Considerable competitive advantage can be drawn from this approach during tendering.

Despite of the obvious benefits which can be drawn from modern computation methods as for example from Finite Element Models by WITTKE (2000), rigid body analysis of underground openings and slopes remains an indispensable tool for risk identification at least in those cases where secondary stress levels are below the strength of a rock mass.

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