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Excavation and primary lining design for tunnel in complex karst geotechnical conditions

Conception de l'excavation et du revêtement primaire d'un tunnel dans des conditions géotechniques complexes du karst

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ABSTRACT: In this paper appropriate excavation and support lining systems are discussed for the 1,5 km long tunnel in karst. Karst formations are made of fractured and friable, occasionally clayey Triassic limestones. Tunnel is in very severe and complex geotechnical conditions, foremost it is not possible to fully explore all karst phenomena prior to the construction (cavities, sinkholes etc.). Tunnel set-out is characterized by the proximity of a grouting gallery of the existing hydropower dam, therefore, conventional excavation techniques (drill and blast) could not be employed on subject section. Farther, tunnel goes through conditions of moderate topographic relief (high overburden), as well as low relief (overburden) covered by thick scree slope, above the crown. Near the exit, tunnel is intersecting with existing unlined tunnels, creating high temporary cuttings which are influenced by snow avalanches and rockfalls, and unfriendly increased tunnel openings on the places of intersection. All through the design, it was tried to identify all hazards and risks in the best possible way, and as outcome typical primarylining, as well as designing philosophy are presented.

RÉSUMÉ: Cet article examine les systèmes d'excavation et de soutien de la voûte appropriés pour un tunnel de 1,5km creusé dans le karst. Les formations karstiques se composent de calcaire triassique fracturé et friable, parfois argileux. Le tunnel se situe dans un environnement géotechnique difficile et complexe, du fait avant tout de l'impossibilité d'explorer l'ensemble des phénomènes karstiques en amont de la construction (cavités, gouffres etc). La situation du tunnel est caractérisée par la proximité d'une gallerie de jointement d'une centrale hydroélectrique ce qui exclut l'utilisation des techniques d'excavation traditionnelles (percement et soufflement) sur le tronçon concerné. De plus le tunnel traverse des zones de relief topographique modéré (fortes surcharges) ainsi que des zones de relief bas (surcharges) couvert d'une épaisse pente d'éboulis au dessus du sommet. Près de la sortie le tunel est entrecoupé de tunnels existants non repérés qui créent d'importantes césures temporaires influencées par les avalanches et les chuttes de pierres ainsi que des ouvertures défavorablement accrues à l'endroit des intersections. Tout au long da la conception nous avons tenté d'identifier tous les dangers et les risques de la meilleure manière possible et en conséquence nous présentons une voûte primaire type ainsi que la démarche de conception.

Keywords: support linning; secondary linning; karst; tunneling

1 INTRODACTION

The road section between Pluzine and Scepan Polje is part of the key road direction and it is a shortest connection between Albania, Montenegro and Bosnia and Herzegovina and Central Europe via Corridor Vc.

The current road goes through Canyon of Piva, above artificial Piva lake. The lake was formed by baffling Piva River in 1975, by constructing high hydroelectric power dam (Mratinje Dam), and flooding part of the canyon. The Dam is arched, made of concrete, with the height of 220 m, and the length of 261 m, with the usable volume of about 800 million cubic meters. The Dam is one of the highest in Europe. The currant road was initial build for the construction purposes of the hydropower dam and later upgrade to serve as primary route – trunk road. One of the main features of the road is that it crosses the Mratinje Dam in its crown.

Current road is on some of the sections, with geometric elements corresponding to a maximum speed of 30 km/h, while slopes are made of scree deposits predisposed to frequent rockfalls and landslides causing road closures. Tunnels are mainly without any lining and of insufficient clearance gauge and road geometric elemetnts in order to meet the national and EU standards.

In order to meet these requirements and standards, significant reconstruction works are required. Current road located in the area of the Dam will be headed with the new long tunnel through the mountain just after the Dam (on the right flank of the Dam).

2 GEOLOGICAL SETTING

2.1 General

The partition profile of the Dam is located on the Piva River. In terms of geomorphological characteristics, the section of the canyon at the dam and tunnel site has the shape of the letter V with very steep and vertical valley sides. The entire right bank is formed by massive Triassic limestone showing non - uniform tectonic disturbance. According to their origins, limestones are of reef origin and it is intensely affected by the significant tectonic movements in the geological history.

In the regional sense, the limestone rock masses of the site belong to the synclinal part of the large anticline (called crkvičke), which extends NW - SE, that is, in the Dinaric direction. Some local tectonic discontinuities are long and deep, and there are numerous smaller discontinuities in the weathering zone, most of which are filled with fine clayey detritus.

The slope of the canyon has a rather uneven relief, typical of thick bedded to massive limestone affected above-mentioned tectonics movements, as well as the action of frost, precipitation and insolation. As a result talus slopes occurs in many places, covering the unevenness of the original relief. The talus thickness, however, increases progressively down slope, from 2 - 3 m to 20 m at the foot.

The sedimentology investigations showed that dolomite limestones are predominantly present, and sometimes they are recrystallized with the content of irregular dolomite crystals. The carbonate substance is usually irregularly disturbed.

The significant jointing of the limestone and the presence of several fracture, as well as the carbonate composition of these rocks, enabled very favorable development of the karst and chemical erosion. Karst erosion is lowered in the area of tunnel (the right bank of the Dam) below the present riverbed and have especially devolved along larger breaks and faults, conditioned by very deep terminal of the river drain and by high water discharge (gradients) from the hinterland. The immediacy of the Mratinje anticlinal and small river basin in the hinterland of the Dam effected to karst erosion lowering. Classic karst phenomena such as large and periodic springs, caves, shelters, etc. are present and distinguished.

2.2 Hydrogeology

Hydrogeology of the canyon differ by hydrogeological characteristics. The Triassic limestone of the right - hand slope has fissure and cavity porosity and functions as an aquifer, with a conduction zone above and a storage zone below the level of the Piva. The aquifer is of the diffuse type, and discharges via a karst spring at the foot of the slope a few hundred meters. Certainly, the position and the regime of the ground waters is generally conditioned by the geology of the terrain, and by hydrogeological properties of the rock mass, and within this, by the regime of the ground waters in the hinterland of the reservoir, as well by the precipitation's regime. The level of established aquifer fluctuates depending on the changes of the water levels in the Dam reservoir. If the reservoir is kept at the same level for a longer period, the equilibrium between the water levels in the reservoir and the ground water inflow establishes. There are large differences of the levels of water in the rock massive, downstream and upstream, in the immediate vicinity of the Dam. Limestone rocks are with poor water permeability features, 0.1 to 5 Lu, whereas the interval of 5 – 10 Lu and 10 – 20 Lu occurs sporadically.

On the right bank of the Dam, there is a access tunnel (grouting gallery) which was used for construction of the piles in the right abutment. The access tunnel passes the grout curtain with downstream inclination. The continuously water flow between the concrete and the rock was observed in the access tunnel, supplying the massif downstream of the curtain. Multiyear monitoring of the water levels in piezometers around the abutment, including trace investigations, are showing that access tunnels has influenced on high water level in the area.

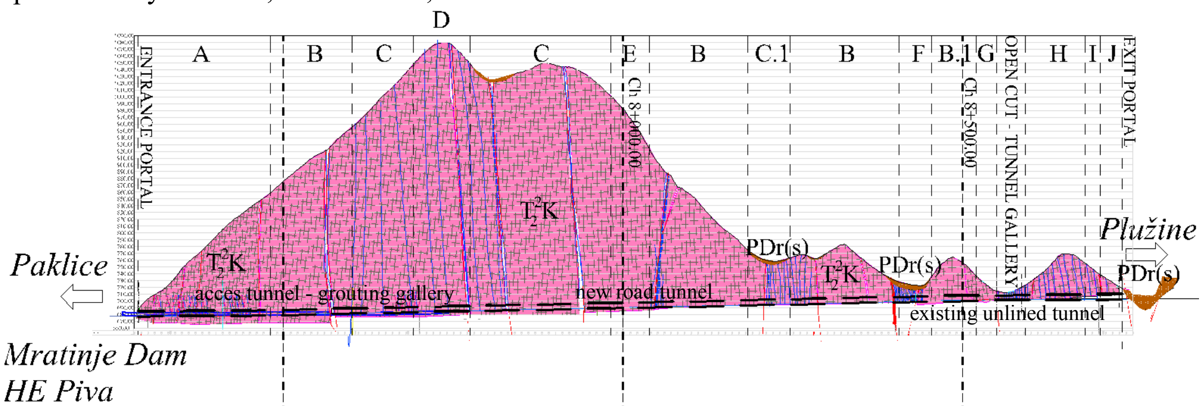


Figure 1. Longitudinal geological section of the new tunnel just after the Mratinje Dam

3 TUNNEL DESIGN - EXCAVATION AND SUPPORT (PRIMARY LINING)

3.1 Basic data on the tunnel

The new tunnel is designed as a single – tube tunnel for bi - directional traffic and a maximal premissible speed of 80 km/h. Tunnel comprises of two 3.25 m wide traffic lanes and two emergancy walkways on both sides of a tube. The basic clearance height above carriagaway is 4.70 m, and height clearance over the emergancy walkways are min. 2.0 m. The theoretical useful cross-sectional area of a tube is governed by a horizontal curve radius and yields A_1= 51.72 m^2 or A_2= 56.40 m^2, depending of the tunnel structure type (Type 1, R_1 = 4.85 m / Type 2, R_2 = 5.15 m. Tunnel is with a left – hand horizontal curve with the radius of 350 m, on the beginning of a tube. The rest of a tunnel is in straight direction. The vertical alignment of tube follows a concave curve with the vert. radius 10,000 m and with max uphill gradient of 2.1 %.

Tunnel is of a horseshoe shape with a max. excavation span of 11.5 m and height of approximately 8.7 m. At the lay – byes, tube is with a span of 17.0 m and 9.45 m height.

Apart from typical cross sections, in order to satisfy tunnel safety measures, there are special tunnel sections which contains lay – by niches, emergancy call niches, fire fighting niches, drainage cleaning niches and niches for electrical and comunication equipment.

A double – sided emergency lay – by niche are provided at the middle of the tube. The lay – by tunnel section is 56 m long. The total useful cross-sectional area of a tube on lay – byes is 95.28 m^2.

Emergancy call niches are provided in the tunnel, alternating, on each side, every 144 m.

Fire figting niches with hydrants and water supply are installed only on the left (hill) side of tube, every 144m.

The drainage cleaning niches are designed on both side, and allows entry to drainage manholes and side drainage pipes Ø160 mm.

There are complete 6 niches for electrical and comunication equipment. Two niches are in the lay – bys and on each side. Additional 3 niches are along the tube and with the identical sizes as emergancy niches. In the vicinity of portals, 2 extra niches are designed with larger size of 2.5 x 5.2 m. Single niche for communication equipment is designed near the middle of a tube with the size of 3.0 x 5.2 m.

There are 2 tunnels which are part of escape routs and allows pedestrian escape in case of a fire to the outside. The basic clearance is 2.25 x 2.25 m, and with useful cross section area of 9.01 m^2. At the jump of each tube pressurized room area of 10 m^2 is provided. Both ends of each room are equipped with double – wing fire – proof door and at the exit to the outside with regular door. First escape tube is 337 m long and exits in the middle of the current tunnel which is omitted by the new road re-aligment. Second escape tube is 56 m long and exits into standing unlined tunnel, near the entrance.

The longitudinal ventilation system in the tube, consists of 16 jet fans 1000 mm diameter, a flow rate Q= 12.17m3/s and drawing force of 133N.

3.2 Excavation and excavation support – tunnel primary lining design

The tunnel is designed to be constructed with the excavation techniques in accordance with the principles of the SCL / NATM .

Two tunnel linings were designed: a primary – initial ground lining consisting of shotcrete, in combination with wire mesh, lattice girder sets or steel arches and rock bolts, and a final lining made of cast – in place reinforced concrete. The inner lining was designed to sustain all internal and external forces without, in this case, considering the bearing capacity of the primary lining.

The excavation will be carried out continuously by roadheaders for hard rock tunneling due to the Dam proximity, or by hydraulic hammer, with a required subdivision of the tunnel cross-section (top heading, drifts, bench, permanent invert). Only escape tunnels and current tunnel widening are requred to drill and blast. To increase the face stability, the tunnel face was supported according to the geotechnical conditions.

The fundamental link between the rock mass and the excavation support classes is the definition of the tunnel behavior classes. The assessment of the tunnel design methodology is presented in the *Figure 2*. Generally, presented procedure covers five steps, and final step of the process is evaluation of the excavation and support measures, and their detail analysis..

The ground quality of the limestone is described earlier and ranges from blocky to very blocky. In order to reasonably estimate the strength parameters of the rock mass, the Hoek – Brown failure criterion (Hoek 2002) linked to the GSI and RMR was used. It was required to consider values of the intact rock properties σ_{ci}, m_i for the rock mass as a single entity. The results from σ_{ci} tests for other objects on the road shows that limestone has wide strength variation of approximately 50 – 80 MPa. No tests were performed on the thin clayey members and fills, but it is estimated to be around 15 – 20 MPa, and around 5 – 15 MPa when shared. In some cases where limestone is calcitic the intact rock is higher (100 – 120 MPa). The properties of the rock joints were based on the macroscopic of the material in the field, since no direct tests were performed, as well on the earlier research in the trial access galleries for the Dam investigations (Ivanović, Kuljundžić 1970). The joints and beddings planes are generally rough to smooth, but when they are interlayered by marlstones or they are shared, they are smooth or slickenside. The other joints are rough to smooth, while their apertures are generally tight without or with clay/silt infilling, therefore they are with strong strength properties. On the rock slope outcrops there is no visible sign of weathering at the rock joint walls. Along shear zones and faults, discontinuity planes have clayey coatings and they might be slippery. However, these zones are thin and are tight to partly open. It has been said earlier that there are evidence of occasional water seepage and dripping along the joints, however, groundwater cannot flow continuously with the uniform level within the entire tube length. Along fault and shearing zones, a local dissolution occurs, as well as seepage due to the blockage of water by the rock. In this case, water is trapped inside the rock and could pressurize rock blocks.

The estimation of the ground conditions refers to the rock ground behavior due to the excavation of the tunnel without any support measures. The key factor of influence are: tunnel geometry, initial stress condition, boundary conditions, water condition and the orientation of the discontinuities (Goricki 2003). Total of 9 rock ground condition are distinguished. Bulky amount of ground conditions are result of the unfavorable boundary conditions such as: vicinity of the grouting gallery, enlarged tube section on the lay-bys, immediacy of the standing unlined tunnel to the new tube or due to settings with the insufficient rock thickness and steep slope, or else, tunneling below the scree deposits and fault, etc. A selection of typical ground conditions are presented in the *Figure 3*.

Geotechnical analysis of the tunnel primary support were completed using the software code Phase^2, considering stress and strain states that develops in the rock around a tunnel. Rock mass is assumed isotropic and homogeneous, i.e failures are not controlled by major structural discontinuities. To determine rock load acting on a support, independent analysis of the rock and tunnel support were performed considering ground response due to advancing tunnel face (convergence – confinement method).

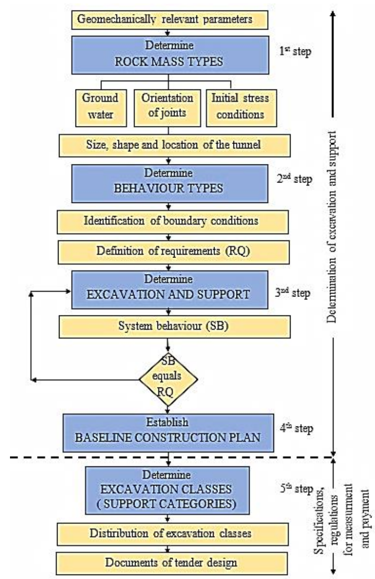
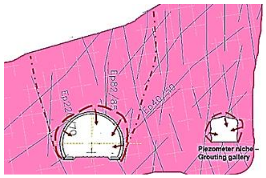
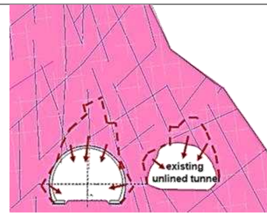
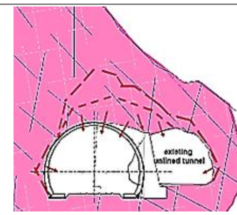
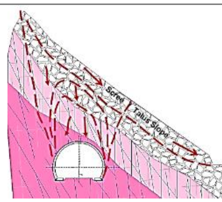
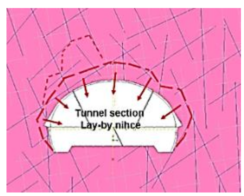
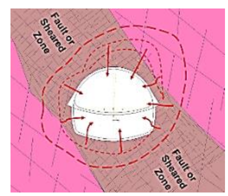


Figure 2. Tunnel design methodology (ÖGG Österreichische Gesellschaft für Geomechanik 2010)

The resulting loads would be successfully undertaken by well-timed placing of support in order to reduce the displacements to desirable – acceptable limits, and it is based on assumption that support will undertake only fraction of the pressure loads due to stress relaxation. In each support category, the finite element calculations are performed for the section with the distinctive overburden height, and the weight of the ground is assumed to determine the initial stresses, the overburden stress is assumed to be a principal stress component. The factor K_0 for the calculation of the horizontal stress components is estimated to K_0 = 0.80–1.00, depending on the ground conditions.

Table 1. Selection of typical rock ground in situ conditions (ground behavior) around tunnel

Ground type	**Ground behaviour type 1**	**Ground behaviour type 7**	**Ground behaviour type 7**
Support Category	A	B.1	H
Ground behavior due to tunneling	 Shallow overburden, h a combination of low stresses, shear failure in the crown	 Discontinuity controlled overbreaks, gravity induced fallings in crown	 Discontinuity controlled overbreaks, induced fallings in the crown due t adjacent tubes
Ground type	**Ground behaviour type 1**	**Ground behaviour type 7**	**Ground behaviour type 7**
Support Category	F	E	Y
Ground behavior due to tunneling	 Shear failures in the crown, due to combination of low stresses and confining stresses or faults	 Discontinuity controlled overbreaks, gravity induced fallings in the crown and sides	 Stresses induced shear failures and gravity controlled failures in the crown and sides, stresses surpasses rock mass strength

Results of the FEM analysis corresponding to the adopted round length and support installation at the predicted distance from the tunnel face, generated stress relaxation around a tunnel in the range of $\lambda = 0.65 - 0.85$ (*Figure 4*). For the large tube section at the location of lay – bye niche (SC E), stress relaxation of $\lambda = 0.2$ was adopted considering high overburden and tunnel geometry. All deformation after stress relaxation builds up stresses in the tunnel primary lining.

Additionally, stability analysis of structural discontinuities - rock wedges, related to the size, shape and orientation of underground opening as well significant discontinuity sets were analyzed by UnWedge. Influence of in-situ stresses are not consider in the analysis, i.e stress induced failures does not takes place. Joints are orientated to cross each other to form wedge as seen in *Figure 3*. Selecting suitable combination of joint sets to wedge overbreaks are based on joint properties, joint sets orientation. The Mohr–Coulomb strength criterion was used to compute the shear strength and each set of discontinuity planes (*Figure 3*).

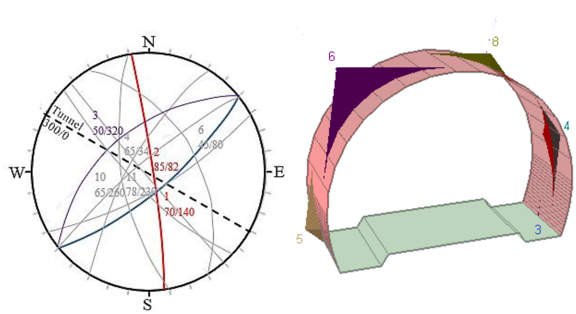


Figure 3. Example of wedge stability analysis for excavation support measure TYPE A (clay infilling c=0-5 kPa, φ=25°)

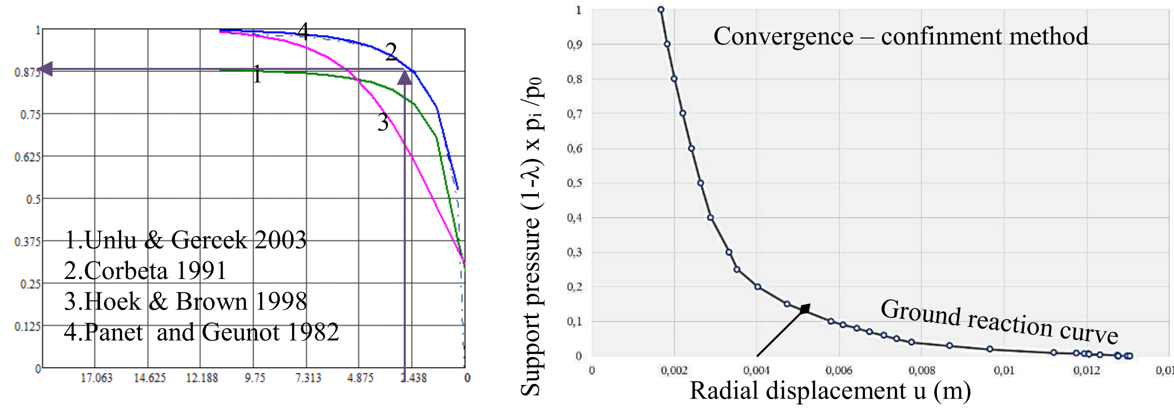


Figure 4. Example of longitudinal displacement along tunnel, and ground - support reaction curves (SC C)

Table 2. Excavation support measures – tunnel primary lining

SC	Ground conditions	Excavation method	Round length	Face measures
A	Immediacy of the grouting gallery	RH / TH + B	TH: 1.5 m; B 3 – 4.5 m	Shotcrete 5 cm,Spiles 36xØ38/19mm,L=4.0m,every rnd.
B	High overburden	RH / TH + B	TH: 2.5 m; B: 2.5 – 5.0 m;	Shotcrete 5 cm
B.1	Adjacent to the current tunnel	RH / TH + B	TH: 1.5 m; B 3 – 4.5 m	Shotcrete 5 cm
C	High overburden,	RH / TH + B	TH: 2.5 m; B: 2.5– 5.0 m;	Shotcrete 5 cm
C.1	High overburden, Faults	RH / TH + B	TH: 2.5 m; B: 2.5 – 5.0m	Shotcrete 5 cm, spiles 36xØ38/19mm,L=4.0m,every rnd.
D	High overburden, Medium stresses	RH / TH + B	TH: 1.5 m; B: 1.5 – 3.0m	Shotcrete 5 cm, spiles 36xØ38/19mm,L=4.0m,every rnd.
E	High overburden, Lay-bye niche	RH / TH +SD + B	TH: 1.5 m; SD:1.5 m B: 2.5 – 5.0m	Shotcrete 5 cm
F	Low confining pressure, shallow tunnel underneath screes	RH / TH + B	TH: 1.5 m; B: 1.5 – 3.0m	Shotcrete 5 cm, face anchors 20 x SDA Ø32mm,L=5.0m,forepoling 37xØ114.3/6.3mm,L=12.0m, every 4th round
G	Low confining pressure due to adjacent of slope	RH / TH + B	TH: 1.5 m; B: 1.5m	Shotcrete 5 cm
H	Low confining pressure immediacy of current tunnel	RH / TH + B	TH: 1.5 m; B: 1.5m	Shotcrete 5 cm
I / J	Enlarging current unlined tunnel	DB / F	F: 2.5 – 3.0 m;	/
Y	High overburden, Medium stresses, Faults, Shear zones	RH / TH + B + Invert	TH: 1.0 m; TINV: 1.0 m B: 2.0 m	Shotcrete 5 cm, face anchors 20 x SDA Ø32mm,L=5.0m,forepoling 37xØ114.3/6.3mm,L=12.0m, every 7th round

SC – Tunnel Support Category; RH – Roadheaders; TH – Top Heading; SD – Side Drifting; B – Benching; F – Full face excavation; INV – Invert; TINV – Top Heading Temporary Invert;

Table 3. Excavation support measures – tunnel primary lining – continue

SC	Shotcrete (thickness in cm)	Rockbolts (in m)	Steel sets	Remarks
A	20 cm, 2x WM T188	SDA Ø32mm,L=4.0m,1.5x1.5m	LG 95/20/30	/
B	10 cm, 1x WM T188	SN Ø25mm,L=4.0m,2.0x2.5 m	/	/
B.1	10 cm, 1x WM T188	SN Ø25mm,L=4.0m, 1,5x1,5m	/	/
C	15 cm, 2x WM T188	SN Ø25mm, L=5.0m,1.5x2.5 m	/	/
C.1	15 cm, 2x WM T188	SN Ø25mm,L=5.0m,1.5x2.5 m	LG 50/20/25	/
D	20 cm, 2x WM T188	SN Ø25mm, L=4.0m 1.5x1.5 m	LG 95/20/30	/
E	20 cm, 2x WM T188	SN Ø25mm, L=6.0m,1.5x1.5 m	HEA 140	*Elephant foot
F	20 cm, 2x WM T188	SDAØ32mm,L=4.0 & L=5.0m, 1.5x1.5m	HEA 140	*Elephant foot
G	10 cm, 1x WM T188	SN Ø25mm, L=4.0m 2.0x1.5 m	/	/
H	20 cm, 2x WM T188	SN Ø25mm, L=4.0 & L=6.0m 2.0x1.5 m	LG 95/20/30	/
I / J	10 cm, 1x WM T188	SN Ø25mm, L=4.0m 2.5x2.5 m, only in tunnel crown	/	/
Y	20 cm, 2x WM T188	SDA Ø32mm,L=9.0m,2.0x1.0m	HEA 220	*Elephant foot *Temporary invert 30 *Permanent invert 50cm

SC – Tunnel Support Category; WM – Wire mesh; LG – Lattice girders;SDA – Selfdrilling anchors;

4 CONCLUSIONS

In this paper, design philosophy used for the tunneling in complex rock ground conditions has been contrasted. Additional, some of the factors affecting the behaviour of the ground and excavation were also identified through stress-strain states analysis and stability analysis of structural discontinuities. All excavation support measures are calculated for the predicted ground conditions in order to contain and control the explicit failure modes upon excavation.

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