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Budarhals HEP – Headrace tunnel excavation – a project review
Budarhals HEP – Excavation du tunnel de chute – examen du projet

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ABSTRACT: The Budarhals HEP 95 MW, in the lower highlands of southern Iceland, is harnessing about 40 m head in a 4 km long 140 m2 in cross section elliptical shaped headrace tunnel. The rock mass on the tunnel route consists of 1 to 2-million-year-old igneous and sedimentary rock formations underlaying hiatus and much younger and highly permeable moberg formation. Interpretation of extensive geological exploration revealed several different geological formations on the tunnel route of different geotechnical properties. The actual geological profile did only to a minor extent vary from the geological tender profile. However, unexpected difficulties were encountered. Statistical analyses and comparison to older comparable projects were performed on the quantity of installed rock support, and evaluation made on the excavation progress advance through different geological formations. This paper summarizes the result of rock mechanical testing, interpretation on deformation measurement and the as-built outcome as compared to similar data from other underground excavation.

RÉSUMÉ: La centrale hydroélectrique de Budarhals, 95 MW, est situé dans les hauts plateaux du sud de l'Islande, et exploite environ 40m de chute dans un tunnel de forme elliptique d´une surface de 140m2 et de 4 km de long. La masse rocheuse sur le tracé du tunnel est constituée de formations de roches ignées et sédimentaires vieilles de 1 à 2 millions d'années sous-jacentes à un hiatus et à une formation de hyaloclastite beaucoup plus jeune et très perméable. L'interprétation de l'exploration géologique approfondie a révélé plusieurs formations géologiques différentes sur le tracé du tunnel de propriétés géotechniques différentes. Le profil géologique réel ne diffère que dans une faible mesure du profil dans les documents d´appel d´offres. Cependant, des difficultés inattendues ont été rencontrées. Cet article résume les résultats des essais de mécanique des roches, de l'interprétation de la mesure de la déformation et des résultats de l'état tel que construit, par rapport à des données similaires provenant d'autres excavations souterraines.

Keywords: Budarhals HEP, tunnel, rock support, lesson learned

1 THE PROJECT

The Budarhals HEP 95 MW is in the lower highlands of southern Iceland harnessing about 280 m^3/s design discharge and 40 m head between two older power stations; Hrauneyjafoss 210 MW HEP and Sultartangi 120 MW HEP. This project was under construction between the years 2010 – 2013 and started operation late February 2014.

The 4 km long and over 140 m^2 in cross section elliptical shaped headrace tunnel, with the height of 15 m and maximum width of approximately 12 m, was excavated through the Budarhals mountain ridge, between the intake reservoir,

Sporöðldulón, and the Sultartangi HEP reservoir, Sultartangalón, see figure 1.

This shape of the tunnel was evaluated the most favourable for the anticipated stress condition in the rock and hydraulic conditions by limit headlosses while simultaneously optimising the rock support efficiency and limit the quantity, emphasising on the top-heading´s support. A similar shaped profile was used in the Sultartangi Hydropower Plant headrace tunnel.

Figure 1. Location map, headrace tunnel alignment

2 GEOLOGY

The rock mass on the tunnel route consists of 1 to 2-million-year-old igneous and sedimentary rock formations underlaying younger and by two to three orders of magnitude higher permeable moberg (pillow lava) formation, separated by a hiatus of about 0,5 million years (Hönnun, 2001).

Extensive geological exploration revealed several different geological formations, of different geotechnical properties, to be encountered on the tunnel route, see figure 2. This included among other three different types of basalt; fractured tholeiite basalt, large columnar porphyritic basalt and geothermally altered olivine basalt, rhyolite and sedimentary rock, mainly sandstone and conglomerate.

Deep under the highest part of the mountain is a highly altered rock mass containing swelling clay minerals such as smectite. This rock mass has in places disintegrated into almost soil like material and would require full concrete lining in a large tunnel. Therefore, to avoid the worst part of this area, the tunnel alignment was bended away from straight line, see figure 1.

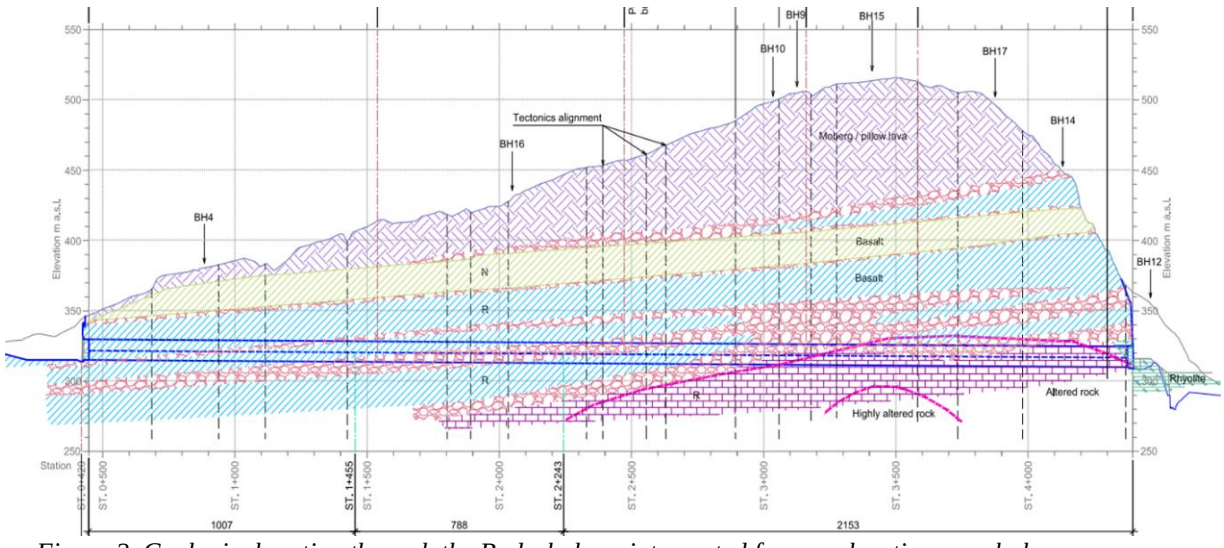

Figure 2. Geological section through the Budarhals as interpreted from exploration core holes.

2.1 Tectonics

At exploration stage, the strike and frequency of tectonic lineaments was mapped from aerial photos, plotted on the geological section model as single broken vertical line, see fig.2

During the underground excavation, tectonic displacement faults consist of few meters up to tens of meters wide influence zone surrounding the fault´s central zone where iterating displacement had occurred.

In all, 29 fault zones were mapped with average length of the influence zone of 15 m, thus about 11 % (450m) of the total tunnel length, see figure 3.

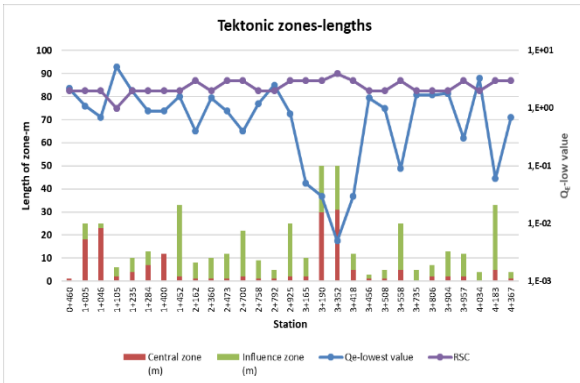

Figure 3. Analyses of the tectonic zones crossing the tunnel route (red is central zone, green influence zone).

Location of some of these faults and fractures coincide with predicted, but other not, thus being older than the overlaying rock. Movement was mostly normal of up to 10m, however reverse and transverse were mapped of up 4m displacement.

2.2 Water ingress

Based on permeability tests in boreholes and field mapping of tectonic lineaments it was evaluated that water ingress would in general be relative limited mainly associated with tectonic fractures and dikes, see figure 4.

Water inflow into the tunnel was expected to diminish with time if faults would not extend up into the highly permeable pillow lava formation above the hiatus, such conditions could be consistent for long time and request extensive grouting measures. To minimize the risk of failure caused by sudden ingress, routine probing was executed.

Figure 4. Inflow of water from fault plane 5m ahead.

2.3 Rock mass qualification

Based on geotechnical test results and rock mass quality measurements (Q-values) on borehole cores it was estimated than 85 to 90% of the rock mass quality on the tunnel route would be as Q > 1, but in the outer flank of the alteration zone and in fault zones, the Q-values in the range from 0,01 to 1 was expected.

Uniaxial strength of the unaltered basalt was measured over 100 MPa, on unaltered sedimentary rock between 30 and 50 MPa and less than 30 MPa for altered rock depending rock type and degree of alteration, in some cases less than 5 MPa.

X-ray analyses of samples of altered rock and fracture fillings in the altered section showed presence of swelling clay, smectite.

3 THE EXCAVATION PROCESS

3.1 Drilling and blasting profile

The tunnel excavation was divided into two sections, top heading and bench, each cross-section of 70 m^2, figures 5 and 6.

The contractor´s method statement revealed number of drill holes in the top heading of 110-125, 51mm wide blast holes and four 105mm cut holes. Drilled depth was assigned 5.3 m and the charging (Q-value) was around 2.3 kg explosive/m^3 rock. (Landsvirkjun, 2016).

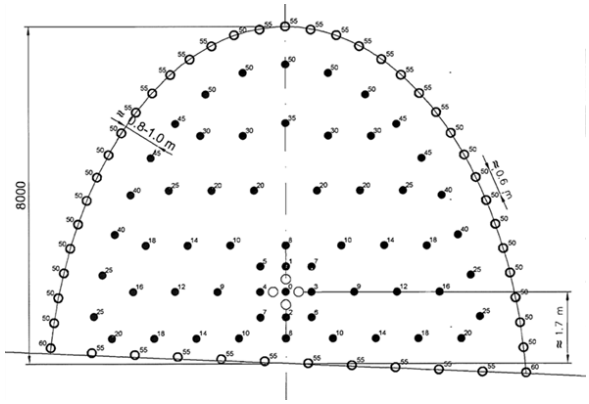

Figure 5. Typical blasting pattern for the top-heading.

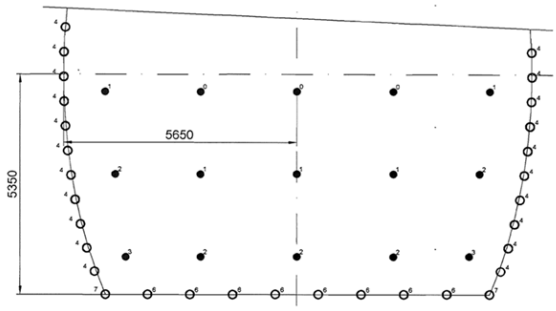

Figure 6. Typical blasting pattern for the bench.

For the bench excavation, number of blast holes of 46 eu., 4.8m long and of 57 mm diameter. Charging (Q-value) was about 1.08 kg/m^3. (Landsvirkjun, 2016).

The contractor´s drilling profile extended about 10 cm outside the excavation design profile, the minimum possible clearance for the drilling arm to avoid underbreaks. The main blast material was a slurry compound.

The tunnel excavation started in late May 2011 from the upstream end and in beginning of October 2011 at the downstream end. Benching started in October 2012 and all excavation was completed in June 2013, followed by move-out of 2 months' work.

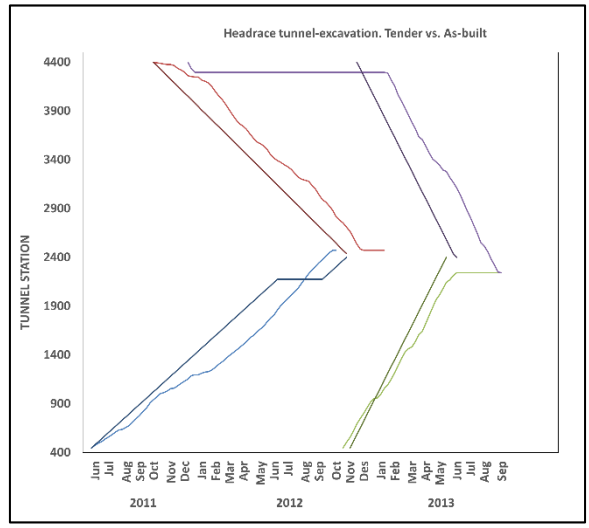

Figure 7. Contractor´s scheduled versus as-built progress of excavation.

3.2 Top heading excavation

3.2.1 Large columnar basalt formation

From the upstream portal and about 1100 m ahead the tunnel is excavated through a 10 to 15 m thick porphyritic basalt formation, with up to 3 m in diameter columns, irregular spaced, undulating joints sometimes filled with up to few centimetres of silt/clay, figure 8.

Figure 8. Large columnar basalt, diameter of 2-3m.

From the start of the tunnel excavation, progress was delayed considerably due to frequent underbreak and re-blasting, see figure 9, and the average progress of excavation was only about 26 m/week.

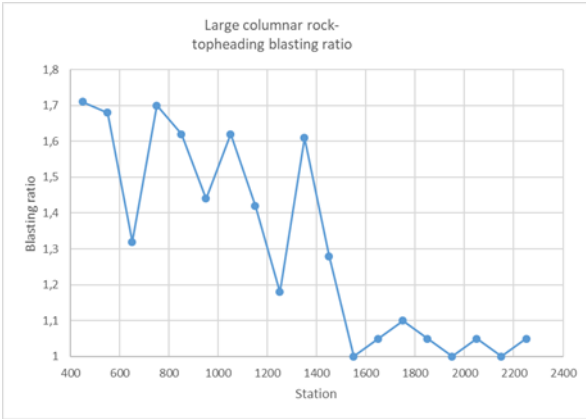

Figure 9. Re-blasting ratio in the first 2 km long large columnar top-heading section. Up to 70% re-blasting occurred in the large columnar basalt, st. 450 to 1550.

3.2.2 Mixed face section; large columnar basalt overlaying sedimentary rock

Underlaying the large columnar basalt is a sedimentary rock containing cobbles and boulders of over 1 m in diameter in sand/gravelly matrix, figure 10, and mixed face condition existed for next about 1,5 km. Excavation through those conditions, underbreak diminished considerably and progress of tunnel excavation was ascending, becoming between 35 to 45 m/week.

3.2.3 Mixed face; altered rock and sedimentary section

Excavation through the alteration zone, excavation progress decreased due to increased rock supporting as per RSC 3 and 4, directing intensive temporary supporting by 6 and 8m spiling followed by sprayed concrete ribs support.

3.3 Bench excavation

The bench excavation profile had about the same cross sectional area, but number of blast holes and charging was about 60% less than for the top-heading section. Maximum weakly advance was thus higher in all different rock conditions, frequently between 60 and 70 m/week.

Figure 10. Mixed face, columnar basalt overlaying, sedimentary rock, st~2.200

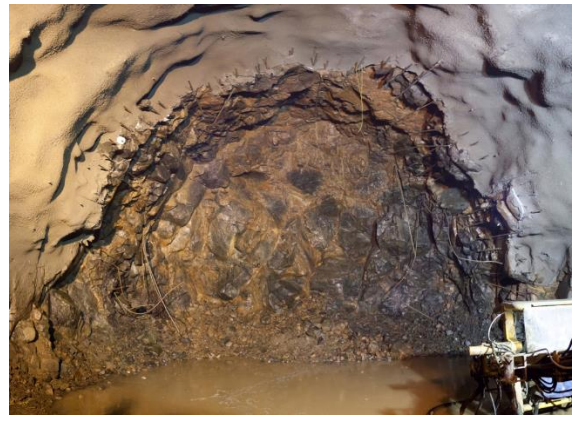

Figure 11. Mixed face, highly altered and unstable intensively weathered rock (rounded boulders) overlaying coarse gravel and boulders. Profile preserved by 6-8m long spiling bolts.

3.4 Rock-mass deformation

To evaluate stability (deformation) of the tunnel cross section, eye-bolts were installed at several tunnel lengths for convergence measurements, and further rod extensometers in two sections in the alteration zone. Horizontal deformation (convergence) was from few millimetres in solid basaltic rock to up to 60 mm in the alteration zone, benching with gallery 90% fully supported.

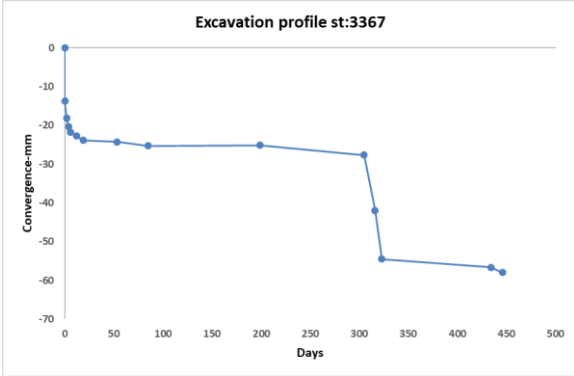

Figure 12. Convergence measurements in the alteration zone, top heading and benching 300 days later.

4 ROCK SUPPORTING

Based on rock mass quality, the rock was divided into 4 rock support classes, figure 13.

Rock support class	Tender documents		As-built		Diff
	m	%	m	%	%
RSC-1	1500	35	949	24	-10,7
RSC-2	2300	53	2417	61	7,9
RSC-3	390	9	449	11	2,3
RSC-4	150	3	157	4	0,5

Figure 13. Rock support class, tender design versus as-built.

The tender bill of quantity estimated 85 % of the tunnel rock mass to be categorized under rock support classes 1 and 2, and accordingly the rock support to be rock bolts and sprayed fibre-concrete.

Due to evaluated swelling clay squeezing rock conditions to be encountered in altered rock formation, requiring more heavy rock support with reinforced sprayed concrete ribs was foreseeable. No steel structures were installed.

4.1 Rock-bolting

In the large columnar basalt, the installed rockbolts were mainly 4 m long with about 2 m spacing in the top heading, and 2,5 to 3 m spacing in bench section.

In the mixed face of basalt and sedimentary rock spacing of bolts in top heading is about 2 m and 3 m in walls in the bench section

In the alteration zone, installed rockbolts are longer and spacing of 1 to 1,5 m in the top heading and 2 m in bench walls, see figure 14. In addition, 8 m long spiling bolts with 0,4 m spacing were installed in the top-heading perimeter in the weakest sections.

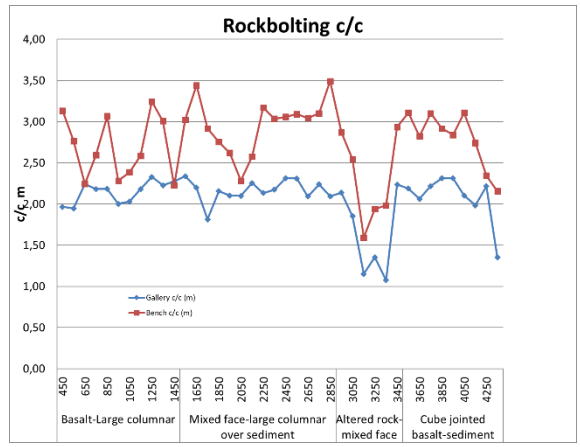

Figure 14. Average spacing of installed rockbolts in top-heading and bench - 100m sections (Landsvirkjun 2016).

About 23.000 rock bolts were estimated in the tender contract, i.e. 5,8 bolts/m tunnel, mostly 3 and 4 m long bolts. In all 31.670 bolts were installed, on average about 7,9 bolts/m tunnel. This is about 37% more rockbolts than anticipated. Bolts were mainly 4 m long and much more spiling bolts were installed than estimated, in the alteration zone and through fault zones.

4.2 Sprayed concrete

In the large columnar basalt, thickness of sprayed concrete was on average of 100 mm in top heading and 60 mm in the bench walls.

In the mixed face, thickness was between 80 and 100 mm in the top heading and 40 to 60 mm in the bench.

In the alteration zone, thickness of sprayed concrete was increased to up to 300 to 400 mm in

the reinforced rib area both in top heading and bench walls.

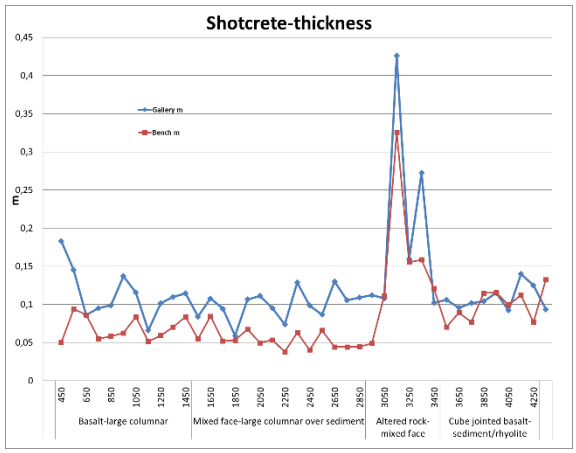

Figure 15. Averaged shotcrete thicknesses in top heading and bench-100m sections.

Figure 16. Sprayed concrete reinforced ribs under construction. Bending of the tunnel alignment ahead.

The tender volume of shotcrete of 12.000 m^3 equated 3 m^3/m tunnel length. Applied sprayed concrete increased by 80% up to 21.600 m^3 equating 5,4 m^3/m tunnel length. This increase was mainly due to underestimate of rock surface roughness (~50%) and shotcrete rebound volume (>10%) in the columnar basalt, and underestimated volume of sprayed concrete applied in the alteration zone.

5 LESSON LEARNED-CONCLUSION

Overall, the tunnel geological profile did only to a minor extent vary from the exploration model, typically due to fluctuation of layers caused by tectonic faulting and layer´s surface irregularity, see figure 17,

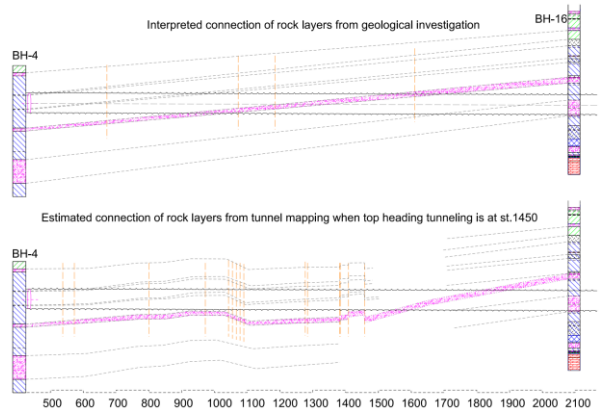

Figure 17. Tender vs. as-built geological model over 1000 m tunnel length in the large columnar section.

5.1 *As-built progress of excavation*

The effective as-built progress of the excavation for the whole tunnel, taken into account all variables, was 34.2 m/week for the top heading and 58.1m/week for the bench. The contractor´s effective estimation in tender was 41,9 m/week for top-heading and 59,5 m/week for the bench, see figure 7, thus 82% and 98% respectively of his estimate.

There are two main reason for lower average excavation progress in the top-heading; intensive underbreak and re-blasting in the large columnar basalt (top-heading) and more rock support the alteration zone than estimated in.

5.2 *Underbreak and re-blast*

From start problems arisen with top-heading excavation in the large columnar basalt, due to undercutting in blasting rounds and recharging and blasting which had considerable effect on the progress of excavation.

The contractor started using slurry explosive in 5,3 m long rounds and as this underbreak was new to him, blasting specialists were called in for consultation on how to resolve this. Number of boreholes, diameter of holes, drilling patterns, types of explosive and charging of holes were

changed without having obvious successive result. In all; 25 drilling and blasting patterns were practised for the top-heading section and the Q-value between 2.3 kg/m^3 and 5 kg/m^3. The separated variables were; number of boreholes, diameter of blasting and relief holes, cut-hole position, and length of pulls. Further, different type of explosive material was tried to minimize the problem as possible.

By statistically comparing results from different approaches in order to minimize the problem, results showed that better results were obtained by reducing the pull length from 5.2 m to 4 m, see figure 18, increase the number of blast holes with less diameter (48 mm) instead of fewer wider holes (64 mm) for the same weight of explosive and use more energy releasing explosives with higher shock wave velocity, such as primer and dynamite along with the slurry.

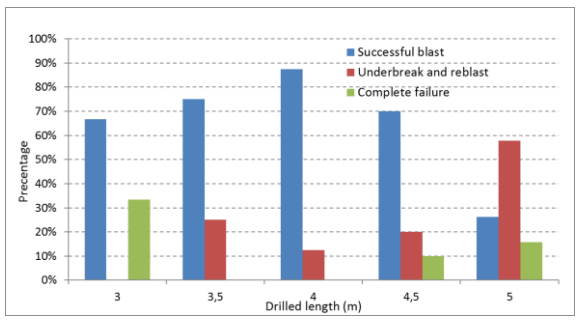

Figure 18 Successful blast (blue), complete failure (green) and underbreak (red).

The most likely reason for the blow-out and underbreak was reflection of the explosive energy wave away from the undulating columnar joint surfaces and energy loss in the joint fill material. Shorter rounds thus reduced this effect and the shock wave from high energy explosives gave better breakage of the columns.

In the civil contract, there was one pay item specified for underground excavation and another for geological overbreak. No pay item or addition was specified for reduced rounds. A lesson learned; to cope with such difficulties where shorter pulls are required and/or give better results, reduced round might be a special work item in the BoQ pending geological conditions specified. A contractor might thus price this accordingly.

5.3 Rock support quantity increase

When quantifying rock support at tender stage, it is important to take into account experienced overbreak in separate types of rock mass, surface roughness and rebound. Frequently, when quantifying tender volume of shotcrete, 35% surface roughness and 5 to 10% rebound is commonly considered covering most conditions. For the large columnar basalt, surface roughness was, however, close to 50%. Further, 10% rebound was frequently the minimum values tested.

The tunnel alignment was bended to avoid the highly altered rock conditions explored in the centre of Budarhals mountain. It was however evaluated acceptable risk to pass through the outer flanks of the alteration zone as being in rock support class 3 and 4 and well known conditions to experienced tunnel engineers.

To be on the save side rock support was though assigned much greater for the top-heading than estimated at tender stage, deemed necessary from Q-value and geotechnical testing. Spiling bolts were installed over about 340 m section and over a 65 m long weakest part reinforced sprayed concrete ribs were installed with about 1 m spacing between 1 m wide ribs, see figure 16.

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