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ABSTRACT

A broad gauge railway line is being constructed by Indian Railways in Himalaya. The total route length is 342kms, out of which about 100km is in tunnels. The tunnelling problem while excavating the Tunnel no.1 of Udhampur-Katra section and being faced currently is discussed in the paper. The D-shaped tunnel passes through thickly bedded, moderately soft, sparsely jointed sandstone, sheared claystones, siltstones and overburden comprising boulders/pebbles in sandy/silty matrix. The support pressure and the deformation were monitored to study the performance of the support system. Due to the presence of swelling minerals in claystone and weak & highly jointed rock formations with high rock cover (313m), the tunnel experienced both swelling and squeezing ground conditions resulting in the buckling of wall supports of steel ribs, cracking of tunnel wall concrete lining at places and floor heaving up to 1.2m. With the deformation of wall supports, the tunnel roof support also deformed. Numerical analysis using FLAC^{3D} has been carried out to study the effectiveness of the support system. The study shows that the tunnel with out any support may have the wall deformations up to 2.76m. On the other hand, with rock bolt and 40cm thick steel fibre reinforced shotcrete (SFRS) support, the wall deformation would reduce to 23cm.

INTRODUCTION

Indian Railways are linking the Kashmir valley in the State of Jammu & Kashmir through Himalayas with a broad gauge railway link which is below snow line making it an all weather route. The total route length is 342kms, out of which about 100km is in tunnels. The ruling gradient is 1 in 100, maximum degree of curvature is restricted to 2.75º. The work from Jammu to Udhampur, 55km long, has already been completed and the section has been opened for running for passenger train since April 2005.

At present, work is in progress on whole Udhampur-Srinagar-Baramulla Rail Link project called as USBRL in short. Udhampur-Katra section is 1st phase of USBRL project which is 25km long and involves construction of 7 tunnels aggregating to 10km. All the tunnels have been bored through. The tunnelling problem while excavating the Tunnel no.1 and being faced currently is discussed in the paper.

GEOLOGY

The tunnels in Udhampur-Katra section fall in Shiwalik Group and Pleistocene to recent deposits. The region is also in the vicinity of a major structural feature, i.e. Murree Thrust. Thus, geologically a considerable length of the tunnel passes through extremely poor tunnelling media.

The Tunnel no. 1 passes through thickly bedded, moderately soft, sparsely jointed sandstone, sheared claystones, siltstones and overburden comprising boulders/pebbles in sandy/silty matrix. The clay mineral analysis of claystone shows the presence of clay minerals like montmorillonite (49%), illite (30%) and kaolinite (21%). Mielenz and King (1995) have reported that all three minerals found in claystone have swelling properties. Further, montmorillonite whose presence was highest has also the highest swelling characteristics.

TUNNEL GEOMETRY, EXCAVATION AND SUPPORTS

Tunnel is D-shaped and 3140m long. The excavated width of tunnel is 6.5m and the height of vertical walls below spring line is 5.0m. The longitudinal section along the tunnel is shown in Fig. 1. The maximum rock cover above the tunnel crown is 313m around chainage 3250m (Fig. 1). Tunnel with high vertical legs at the first instance seems to be unstable shape specially in weak, jointed and sheared rock masses as encountered in the tunnel.

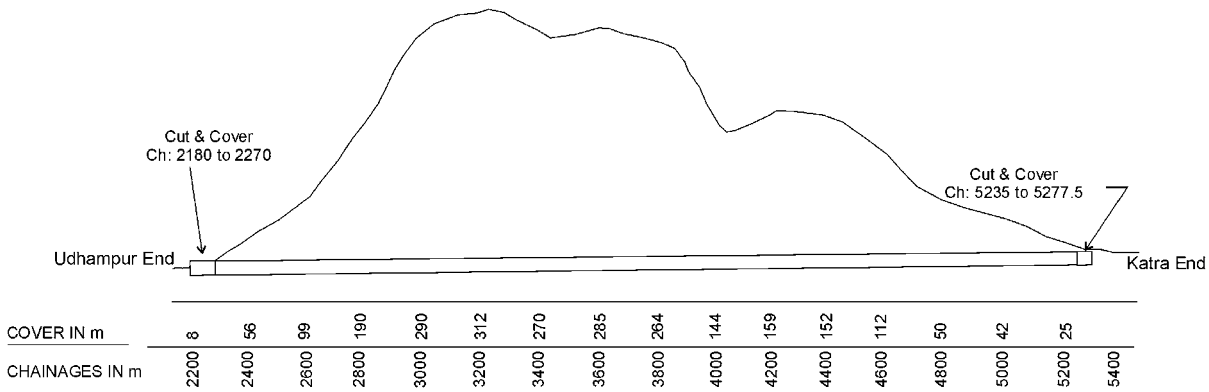

Fig. 1. Longitudinal section of Tunnel 1 from ch. 2180m to 5320m (Sharma and Chopra, 2006).

Table 1. Rock mass quality Q and rock mass number N for claystone

Rock Type	RQD	J_n	J_r	J_a	J_w	N	SRF	Q
Claystone and Siltstones	10-15	12	1.5-2	4	1	0.31-0.62 (0.44)*	5 -10	0.0312-0.125 (0.062)*

* *log average value*

Construction of tunnel was started in June 2000 from the two ends, called as the Udhampur end and the Katra end at chainage (ch.) 2180m and ch. 5320m respectively. The underground excavation was completed in October 2004 with the tunnel from two end meet at ch. 3746m. Most of the underground construction was carried out in two stages, i.e. heading and benching. The primary support system was single rib ISHB 150 @ 500mm/750mm spacing centre to centre with backfill of M10 concrete. With this primary supports the excavation remained continued upto ch. 3420 from Udhampur end and upto ch. 4420 from Katra end. In the central 1000m zone from ch. 3420m to ch. 4420m single support system has been strengthened to double rib support system because of weak rock mass and high order of deformations (Sharma and Chopra, 2006). The double rib system consists of outer rib ISHB 150 @ 500mm/750mm centre to centre and inner rib ISHB 150 @1000mm centre to centre; arch support resting on wall beam of ISHB 200/ISHB 150; laggings of RCC slabs; backfill by M10 concrete and filling between ribs by M20 concrete.

GROUND CONDITIONS

The claystone has been classified using the Q-system of Barton et al. (1974). Qualitatively the claystone is described as sheared and highly jointed with three joint sets and random joints; joints are closely spaced and the walls of joints are slickensided, undulating and smooth; altered joint walls having coating of gouge material. The rock at the time of excavation is generally dry but with passage of time becomes moist. For practical purposes, silt stone can also be grouped in the same class of claystone. Accordingly, the rating of various parameters of Q-system and value of Q for claystones is worked out (Table 1). The value of rock mass number N (defined as the stress free Q, i.e. Q with SRF = 1) has also been given in Table 1. In Table 1 parameters RQD, J_n, J_r, J_a, J_w, SRF and Q are defined by Barton et al. (1974).

Estimation of Ground Conditions

The information on the ground condition is required for selection of excavation method and designing the support system for underground openings.

Non-squeezing and squeezing ground conditions have been predicted by using following equation proposed by Goel et al. (1995) wherein effect of tunnel size has also been considered:

$$H = (275\ N^{0.33})\ B^{-0.1} \quad \text{metres} \qquad (1)$$

Where H is tunnel depth or overburden in metres, N is rock mass number (stress free Q, i.e Q with SRF =1), and B is tunnel span or diameter in metres.

Equation 1 implies that for a squeezing ground condition to occur, $H > (275.N^{0.33})B^{-0.1}$ metres and for a non-squeezing ground condition, $H < (275\ N^{0.33})\ B^{-0.1}$ metres.

Table 2. Prediction of degree of squeezing using rock mass number N (Goel et al., 1995)

S. No.	Degree of Squeezing	Correlations for Predicting Degree of Squeezing	Critical Tunnel depth H for N=0.44 and B = 6.5m
1.	Mild squeezing (u_a/a =1-3%)	$275N^{0.33}.B^{-0.1}<H<450N^{0.33}.B^{-0.1}$ and $J_r/J_a<0.5$	170 < H <280
2.	Moderate squeezing (u_a/a =3-5%)	$450N^{0.33}.B^{-0.1}<H<630N^{0.33}.B^{-0.1}$ and $J_r/J_a<0.5$	280 < H < 395
3.	High squeezing (u_a/a > 5%)	$H>630N^{0.33}.B^{-0.1}$ and $J_r/J_a<0.25$	395 < H

Notations: N = Rock mass number (Q with SRF=1); B = Tunnel width in metres; H = Tunnel depth in metres; u_a = radial tunnel deformation, a = tunnel radius in metres, J_r = Barton's joint roughness number & J_a = Barton's joint alteration number.

For rock mass number N = 0.44 and excavated tunnel span B = 6.5m, from Eq. 1 the minimum depth for squeezing to occur is 170m. Accordingly, the tunnel depths for various degree of squeezing are shown in Table 2.

The maximum tunnel depth is 313m at Ch. 3250m. Table 2 shows that the tunnel shall experience the moderate squeezing ground condition under the cover of around 313m. In addition swelling conditions were also encountered because of the presence of swelling minerals in claystones.

PRESSURES AND DEFORMATIONS IN TUNNEL

The support pressure estimated from the empirical approach of Goel et al. (1995) was 0.84 MPa in tunnel having rock cover of 300m and allowing the tunnel deformation of 4 per cent (27cm). The high vertical wall in weak rock is also adding to the instability of the tunnel. Hence it was cautioned that the construction shall be carried out with full precaution.

On the basis of above empirical approach, the steel rib support of maximum capacity 0.84 MPa with loose muck backfill which can allow 15-20 cm (3-4 % of tunnel size) of controlled radial rock deformation has been suggested to use as primary support during the excavation of the tunnel. Idea behind this was that the loose backfill will absorb the rock deformations and shall reduce the load on the steel ribs. Accordingly, the steel rib supports with loose muck backfill have been installed in the tunnel and the support behaviour has been monitored by installing the load cells and the closure points.

MONITORING OF ROCK AND SUPPORT BEHAVIOUR

Development of rock load on steel ribs and rib deformation were monitored. The load cells and the closure points for measuring deformation are installed as shown in Fig. 2 on the outer rib. Figure 3 shows the position of face and bench at the time of installation of instruments on December 13, 2003.

The data of the two load cells installed at ch. 3461m is plotted against time and shown in Fig. 4. The rib deformation/closure at ch. 3461m and the face & bench advance is also shown in Fig. 4.

Excavation work of face and bench was continuously going on after installation of instruments. Figure 4 shows that there is almost no development of load up to 20 days. The face advance effect was not shown by the load cells for 20 days. This is because the wall beam has also moved under the influence of bench excavation and thus the wall beam could not provide the desirable reaction. Subsequently, it is instructed to ensure the stability of wall beam especially in the heading and benching zone of tunnelling.

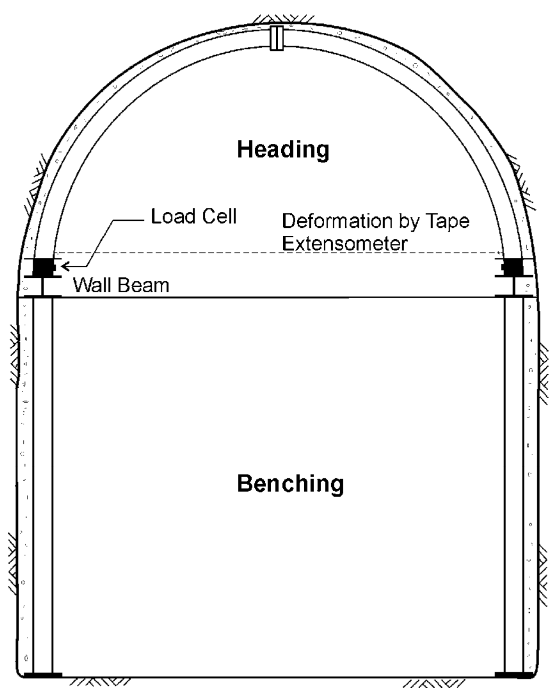

Fig. 2. Section showing position of load cells and the closure points, ch. 3461m, Udhampur end.

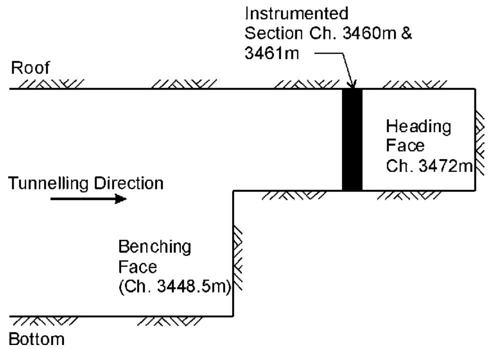

Fig. 3. L-section showing position of instrumented section with heading and benching faces, Udhampur end.

The development of load started after 20 days. Initially the rate of load development was high but with time it has reduced. After about 140 days of observations, the rate of load development has almost stabilized (Fig. 4). The load shown by the left and right load cells is around 58T and 70T respectively. Considering the inner span of steel rib as 5.50m and rib spacing as 1.0m, the support pressure works out from the load is between 2.3 to 2.5 kg/cm2 (0.23 to 0.25MPa).

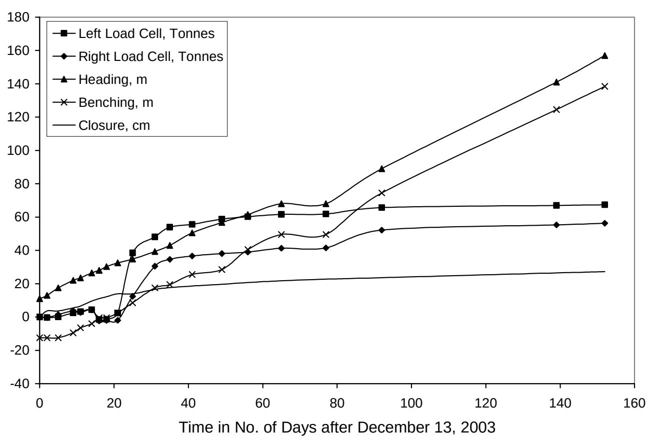

Fig. 4. Time vs. load, closure, face advance and bench advance, ch. 3461m, Udhampur end.

Figure 4 also shows the plot between time after December 13, 2003 and the deformation or closure. Initially the deformations were insignificant, but after 10 days it increases and continues to increase gradually till the last observation (Fig. 4). The maximum deformation is about 25cm in about 150 days, which is about 4.5% of tunnel size (inner tunnel width 5.5m). The increase in load and deformation remains continued even when the face was about 90m away from the instrumented section, i.e. about 15 time the tunnel width.

The suggested primary flexible support system has been adopted in the tunnel without any difficulty and worked satisfactorily.

It was also suggested that in case the rib deformations are increasing at an alarming rate, the rib support shall be strengthened by double rib and concreting between the outer and inner rib.

TUNNEL PROBLEMS

Higher order of deformation because of swelling of clay minerals is the cause of concern in the tunnel. This has led to deformation of steel rib supports at number of locations in the tunnel. The deformations remained continued even after 12 to 18 months of excavation. Table 3 gives the status of deformations of tunnel at various locations. It can be seen in Table 3 that deformations are much more than the estimated/expected values of deformations and as such even the concrete lining has deformed and cracked between ch. 4400 and 4900m. Figure 5 shows the plot between tunnel wall deformation and time. It can be seen in the Fig. 5 that deformation rate has increased rapidly before the collapse occurred on 30.6.06 at ch. 4831m.

Table 3. Measured tunnel deformations at various locations in Tunnel no. 1 (Sharma and Chopra, 2006).

Chainage, m	Rock Cover, m	Measured tunnel deformation, mm
2180–2600	<150	< 65
2600–2900	150 – 280	Max. upto 400
2900–3400	280 – 313	500–580
3400–3800	280 – 31	220–740
3800–4000	150 – 280	180–340
4000–4400	Approx. 150	250–320
4400–4900	150–50	Upto 740mm before lining & further 210mm after lining , i.e. lining also cracked
4900–5320	< 50	< 65

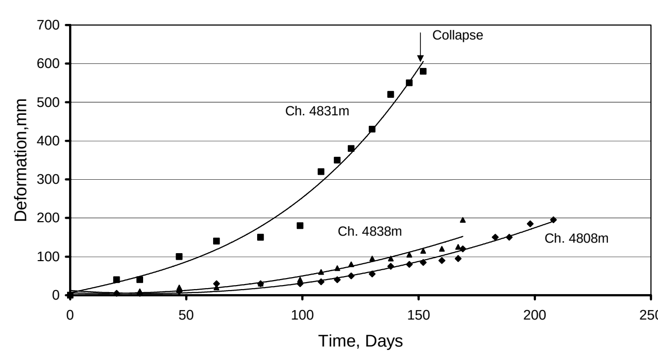

Fig.5. Time – deformation plot at chainages 4808, 4831 & 4838m, Tunnel 1.

In fact the tunnel deformations from all the sides were observed. Even the floor heaving has been observed to the order of 1.2m. The vertical tunnel wall support is attracting maximum bending moment and is therefore working as the weakest link in the support system. The measured deformations are also maximum in the centre portion of the walls. The concrete lining has also started cracking initially in the middle portion of the walls.

Looking into the problems of tunnel support failures, it was decided to re-assess the support system using numerical technique before actually starting the rectification work in the tunnel.

NUMERICAL ANALYSIS

Numerical analysis using FLAC^{3D} of ITASCA, USA has been carried out to study the requirement and effectiveness of the support system.

In Situ Stresses

The in situ stresses vary with depth. In case of rock masses, there are significant horizontal stresses even near ground surface due to the non-uniform cooling of the earth crust. Moreover, the tectonic stresses also affect the in situ stresses significantly. Hoek and Brown (1980) analysed world-wide data on measured in situ stresses. They found that the vertical stress σ_v is approximately equal to the overburden stresses as given in eq. 2, where D is the depth of tunnel below ground surface in meters.

$$\sigma_v = 0.027\, D, \qquad \text{MPa} \qquad (2)$$

The ratio of σ_h / σ_v is denoted by k, for our design purpose, value of k has been taken as 1.0 and 1.5 for estimating the horizontal in situ stresses. Therefore at 300m depth, two stress models have been considered for the analysis (Table 4).

Table 4. In situ stress model in MPa

	k = 1	k = 1.5
σ_v	8.1	8.1
σ_h	8.1	12.15

Rock Properties

The rock properties for the analysis are given in Tables 5. In absence of actual measured values of various rock properties, these values have been judiciously assumed.

Model Dimension

The boundary conditions and the dimensions used for the analysis are given in Fig. 6. The tunnel is modeled as half symmetry geometry. The size of the tunnel is taken as 8.5m (H) x 7m (W).

Table 5. Input rock properties for numerical analysis

Hoek's Geological Strength Index (GSI)	30
Young's Modulus (E), GPa	0.3
Hoek & Brown rock material constant, m_i	7
Hoek & Brown Disturbance factor, D	0.6
Uniaxial compressive strength of intact rock σ_{ci}, MPa	35
Hoek & Brown rock mass constant, m_b	0.2
Hoek & Brown rock mass constant, s	0.0001
Hoek & Brown rock mass constant, a	0.5
Rock density, kg/m^3	2700
Poisson's ratio, ν	0.26

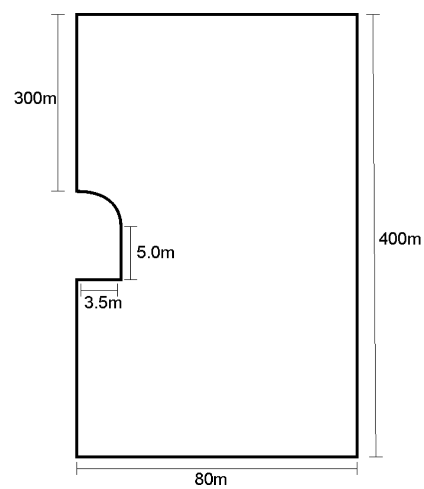

Fig.6. Dimensions and boundaries of the model for numerical analysis.

Modelling of SFRS and Rock Bolt Supports

The primary supports used in the analysis are steel fibre reinforced shotcrete (SFRS), rock bolts and combination of both. The properties of supports used in the analysis is given in Table 6.

Nine tunnel models were prepared for one in situ stress model for various combinations of supports including the tunnel without support. Thus a total of 18 models were analysed for two stress models (k = 1 and 1.5).

Table 6. Support properties of SFRS and rock bolt

Steel Fibre Reinforced Shotcrete (SFRS)	
Young's modulus(E), GPa	15
Poisson's ratio, ν	0.25
Density (ρ), kg/m^3	2500
Cohesion, MPa	0.15
Residual cohesion, MPa	0.1
Tension, MPa	0.05
Friction angle	25º
Rock Bolt	
Bolt length, m	4
Bolt dia., mm	25
Bore hole dia., mm	38
Perimeter, m	0.0785
Area of cross section, m^2	5e-4
Young's Modulus of steel $E_{(steel)}$, GPa	200
Poisson's ratio of steel, ν	0.25
Bond cohesion, MPa	3.8e5
Bond friction Angle	21º

Analysis and Results

As mentioned above a total of 18 models were analysed for the study in two parts i.e. unsupported and supported. Further the supported analysis was divided in two parts, i.e. with SFRS only and SFRS & rock bolts. The results of the analysis have been tabulated in Table 7.

The results indicate that major plastic deformations occur in both supported and unsupported conditions. The deformations are about 23cm after installation of 40cm thick SFRS (Table 7). The effect of rock bolting is also observed to be negligible for all cases of support analysed which may be because the bonding at the interface of rock bolt and rock fails due to the weak rock mass properties.

The results of the analysis are sensitive to the assumed value of input parameters, like in situ stress, etc. But, the analysis clearly shows high order of rock deformations and the requirement of heavy supports to contain the deformations.

Hence it is recommended that the SFRS with high energy absorbing capacity (approximately 1000 joules) shall be applied in the rectification work in collapsed portion is yet to start. To reduce the deformations of the SFRS, its flexural toughness should be high, which can be achieved by using more quantity of standard steel fibres (say around 50 kg/m^3) in SFRS. It is also proposed to get tested the SFRS as per EFNARC suggested method for the above required energy absorption capacity before actually applying in the tunnel.

The past experience in the tunnel shows that 100mm thick SFRS has cracked in the rectified areas. Hence the thickness of SFRS shall be increased to 200mm.

Table 7. Results of numerical analysis

	Deformation, m		Pressure, MPa	
	Roof	Wall	Roof	Wall
Model – I : k = 1.0				
Case I – Unsupported				
	3.651	2.761	0.715	0.026
Case II - Supported				
10 cm	1.642	0.420	0.187	0.610
10cm + RB	0.888	0.403	0.386	0.570
20cm	1.031	0.345	0.379	1.350
20cm + RB	0.697	0.336	0.465	1.330
30cm	0.240	0.292	1.740	1.540
30cm + RB	0.570	0.312	0.666	1.580
40cm	0.543	0.308	0.916	1.660
40cm + RB	0.467	0.299	0.930	1.650
Model – II : k = 1.5				
Case I – Unsupported				
	2.736	2.189	0.037	0.025
Case II – Supported				
10 cm	1.174	0.326	0.169	0.474
10cm + RB	0.677	0.313	0.316	0.462
20cm	0.767	0.263	0.304	1.046
20cm + RB	0.535	0.255	0.370	1.036
30cm	0.198	0.224	1.443	1.203
30cm + RB	0.439	0.238	0.517	1.231
40cm	0.414	0.234	0.707	1.300
40cm + RB	0.360	0.229	0.730	1.280

** RB – Rock Bolt*

The shotcrete and rock bolt shall be applied in steps as follows:

Step 1. First layer of 100mm thick SFRS of energy absorbing capacity of 1000joules. The SFRS should have perfect bond with the rock mass.

Step 2. Installation of resin grouted rock bolts as per design (Table 6).

Step 3. Second layer of 100mm thick SFRS of 1000 joules energy absorbing capacity.

Step 4. Erection of ISHB 200 steel section as per design (To support the arch supports, the steel rib may be installed in the walls before the application of SFRS and rock bolt).

Step 5. Same support and the supporting pattern should be followed for supporting the curved invert.

Step 6. Deformation and stresses of the rectified section should be monitored for assessing the tunnel stability.

Further it is suggested that proper drainage should be provided in the tunnel to prevent seepage due to stagnation of water.

CONCLUSIONS

From the above case history of Tunnel No. 1 of Udhampur-Katra section, following conclusions are drawn.

- The tunnel is facing the problem of squeezing and swelling ground conditions, the swelling condition being more problematic because of the presence of swelling prone clay minerals.
- High order of deformations are measured even after 12-18 months of tunnel excavations leading to the failure of steel rib supports and also the cracking and failure of tunnel lining at places in the tunnel between ch. 3400 and 4900m.
- The numerical analysis shows that the tunnel require heavy supports for stability.

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