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Chenani-Nashri Tunnel, the longest road tunnel in India: the "Himalayan challenge" for design in heterogeneous rock masses

The 9 km long Chenani-Nashri Tunnel, currently under construction, is the longest road tunnel in India and is part of the planned four-lane widening of the NH-1A between Udhampur and Banihal in the state of Jammu and Kashmir. Bypassing the existing NH-1A from km 89 to km 130, the tunnel crosses a sub-Himalayan formation with a maximum overburden of 1,050 m. With an escape tunnel running parallel to the main tunnel, excavation is performed by Leighton-Welspun Contractors using the drill and blast method. Geodata Engineering (GDE) is providing consultancy services for detailed design and construction supervision including 3D-geotechnical monitoring. Back-analyses of already-excavated sections are performed to better understand the behaviour of the heterogeneous rock mass in which the tunnels are excavated. The numerical models are fed with the actual geological and geomechanical conditions encountered during excavation and the monitoring results. The 3D-monitoring system, specially implemented by GDE for this project, has played a key role in understanding the real rock mass behaviour, allowing the highlighting of potential risks, selecting the correct tunnel support class, checking of the effectiveness of countermeasures, identification of the tunnel stretches in which the final lining needs to be reinforced and providing cost-effective solutions to speed-up the construction process.

1 Project description

The Indian Government has entrusted the National Highways Authority of India (NHAI) with the responsibility for the rehabilitation, strengthening and four-lane widening of the Chenani to Nashri section of the NH-1A from km 89 to km 130, the "Highway Project" (Fig. 1), which is the most important and recent Indian project planned to connect the Kashmir valley with the rest of the Indian transportation network.

In addition to the underground works, the project includes surface works comprising a 1.3 km approach road to the south portal and about 0.6 km of road approaching the north portal, two bridges, toll plazas, all project facilities and spoil dumps. The Chenani-Nashri system includes a 9 km long bi-directional (two lane) main tunnel with a separate parallel escape tunnel designed as an emergency access for rescue teams during operation as well as for site traffic during the construction stage. These two tunnels are connected by pedestrian cross-passages every 300 m and vehicular cross-passages at intervals of 1,200 m. For the main tunnel (Fig. 2), the total width of the clearance profile is 11.75 m including the 9.35 m wide paved carriageway and 1.2 m wide walkways on both sides. The tunnel geometry has also been defined to take into account the provision of a fully transverse ventilation system (exhaust air and fresh air ducts are provided). The ventilation slab is supported on corbels in the side walls (cast in one piece with the final lining using a formwork unit equipped with a mobile "shoulder") and is suspended at the crown by epoxy-coated steel anchors embedded in the RC partition wall. The escape tunnel geometry has been defined in conjunction with the contractor on the basis of a detailed study of the available construction equipment such as jumbos, excavators and dumpers. The escape tunnel complies with the required clearance dimensions for the operational stage (5.0 m width) with fully "vehicular" footpaths on both sides. Cross-passages allow the transit of machines and equipment and it was, therefore, possible to use them to open additional excavation faces for the construction of the main tunnel, permitting faster excavation rate of the escape tunnel compared to the main tunnel.

The Chenani-Nashri Tunnel is being excavated by conventional drill and blast method. The excavation face is sub-divided into top heading, bench (right half and left half alternatively) and invert (if necessary). A mechanical excavator will probably to be used in the sectors, such as shear zones, where weak rock mass conditions are expected.

The excavation cross-section of the main tunnel varies from 130 to 170 m^2 depending on the support class. Primary lining includes shotcrete, rock bolts and lattice girders (where required) to stabilize the rock mass immediately after excavation. The final lining (varying in thickness and reinforcement as required) has been designed to withstand the predicted long-term loads and seismic loads, which are not negligible in the vicinity of the portals. Between primary and final lining, a waterproofing membrane with geotextile protective felt is installed from the top of the tunnel down to the concrete foundation beams.

2 Geological context, rock mass classification and design criteria

The project area lies in the western Himalayan region, in the collision belt sector better known as the sub-Himalayas. This tectonic domain is delimited in the south by

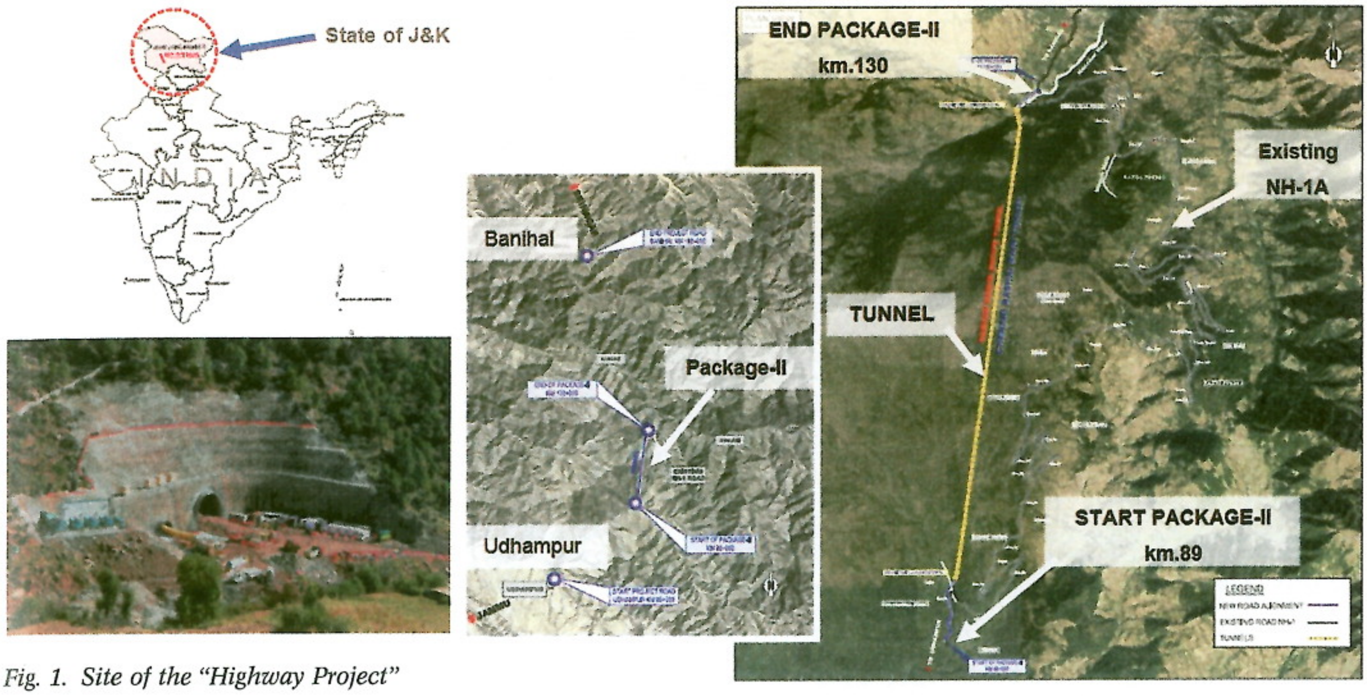


Fig. 1. Site of the "Highway Project"

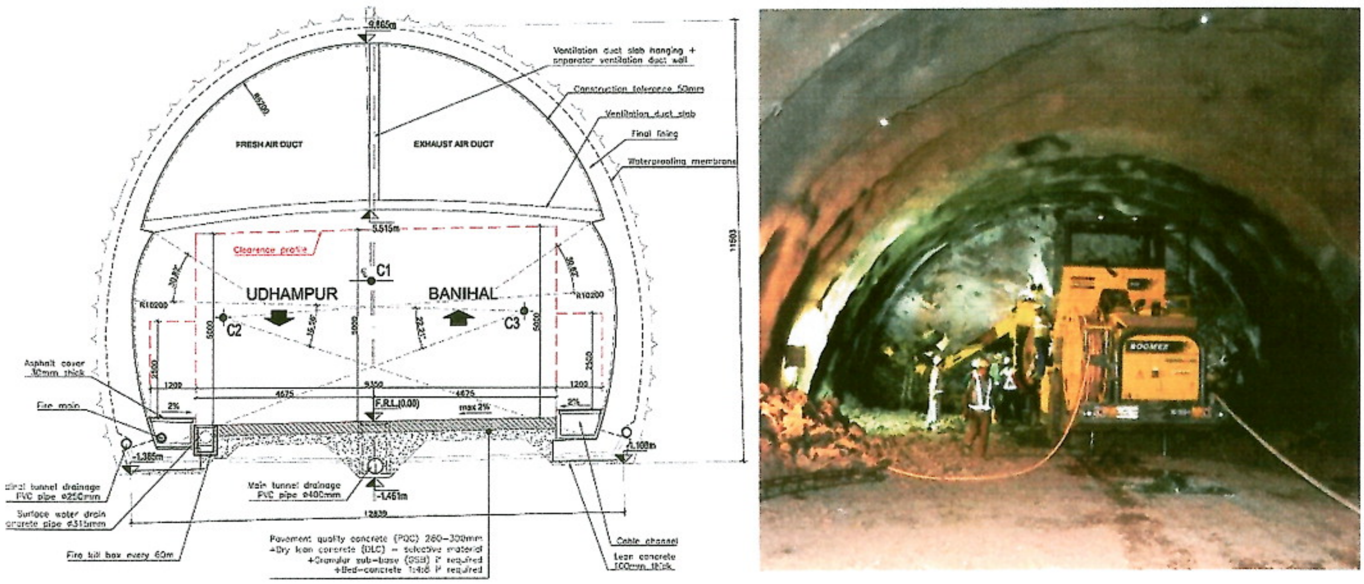
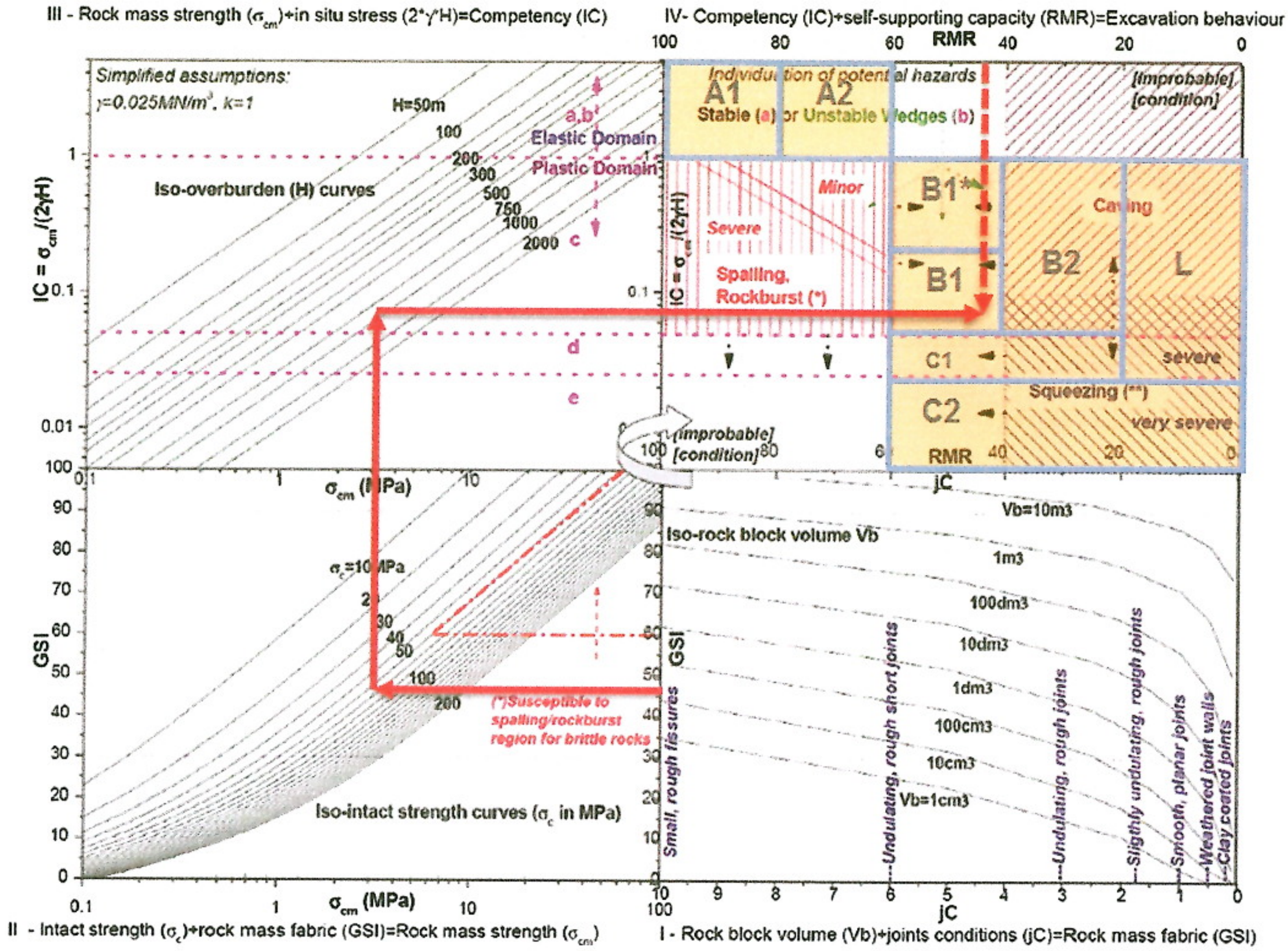


Fig. 2. Typical section of main tunnel and the jumbo used for the heading excavation

the Himalayan main frontal thrust (MFT) and in the north by the main boundary thrust (MBT). The rock masses along the tunnel alignment belong to the Lower Murree Formation which includes a sequence of interbedded sandstone and siltstone/claystone layers (flysch formation) with thickness ranging from a few layers to 10 m. Given the deep overburden, one of the most critical aspects during the design stage was to define the relative pre-dominance of the different lithologies (sandstone, siltstone and claystone) at a tunnel scale. More generally, the design was mainly developed using the analytical probabilistic approach and the expected excavation behaviour was calculated taking into account variability and uncertainty of rock mass properties and in-situ stress. Such a quantitative method allowed the assignment of adequate support types for each geomechanical hazard [1] [6]. The same criterion was followed consistently during construction stage with the systematic application of the so-called "GDE multiple graph" [1] [2] [7], see Fig. 3.

In particular, the field of application of each support class has been reported in the 4th quadrant of the graph (at the top right side), as results from the associated design combination of geomechanical properties. In this way, the correct support section type is directly selected at the tunnel face by assessing the key classification parameters (i.e. GSI, intact rock strength σ_{ci}, in situ stress and RMR). An example related to the case analyzed in the next section is shown in Fig. 3. For this specific case a "fictitious" overburden was considered to simulate the anisotropic state of stress predicted during the design stage and confirmed by the back-analysis. Note that the fictitious overburden determines the same maximum stress derived by the equa-



(*) only for the susceptible region, otherwise the development of plastic region and moderate radial convergences are more probable

(**) depending also from the length of the potential proned zone: given a possible "silo effect", for short zones included in good quality rocks, a caving behaviour it is most likely

Fig. 3. Application of the "GDE multiple graph" at TM 820 [2] [7]

tion $3\sigma_1$-σ_3, where $\sigma_1$and σ_3 are the principal in situ stresses, hence the "fictitious" overburden can be easily calculated as $(3\sigma_1-\sigma_3)/2\gamma$. Moreover, it should be observed that in place of the quantitative assessment of the GSI in the first quadrant of the graph (at the bottom right side), the specific *Hoek* and *Marinos* chart for heterogeneous rock mass [3] [4] [5] was applied and the resulting GSI was directly entered in the second quadrant.

3 Back-analysis of an instability phenomenon during tunnel construction

A local gravitational instability involving the right crown/side wall of the tunnel occurred approximately 820 m from the south portal. Several cracks appeared in the shotcrete lining immediately after primary support installation, showing an unexpected response of the rock mass. The cracks were mainly localized on the right crown/side wall of the tunnel where a claystone/siltstone layer would be constantly intercepted during top heading excavation due to the unfavourable orientation of the flysch strata (almost sub-parallel to the tunnel axis).

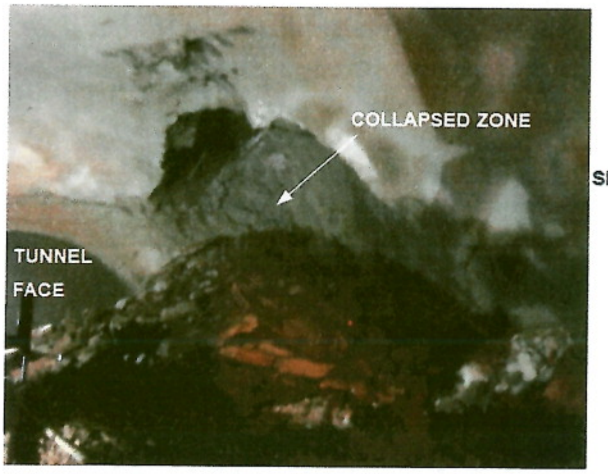
According to the face mapping, the presence of weak interbedded siltstone and claystone layers was affecting the upper right side of the tunnel face. Three sets of prominent joints and a few random joints were detected, which were closely to widely spaced with tight to open apertures. Some of the joints had silty-clay infilling. In that sector, the geomechanical classification of the top-heading excavation initially indicated the requirement of the support class "B1*", with 2.5–3.5 m round length, 150 mm fibre reinforced shotcrete (FRS) and systematic bolting. Nevertheless, as described below, the adjustment of some classification parameters was necessary based on the back-analysis and a different support class was specified.

In order to better understand the causes, which led to the cracks and to the instability, several back-analyses were performed using the recorded geotechnical and geostructural conditions along with the available results of monitoring. The numerical model presented below was defined taking into account the strata orientation shown in the face mapping and the state of stress dependent on the overburden of 350 m. It should be noted that in order to achieve results consistent with the monitoring data, it was necessary to apply an anisotropic state of stress, with the major stress oriented in the same direction as that of the strata.

The key geomechanical parameters for each lithology were calculated using an iterative process and the figures shown in Table 1 were finally selected.

Table 1. Geomechanical parameters adopted in the numerical analysis

Lithology	GSI [-]	UCS [MPa]	m_i [-]	m_b [-]	s [-]	a [-]	E [MPa]	ν [-]
Sandstone	45	100	11	1.5428	0.0022	0.5081	6150	0.25
Sandstone/siltstone	45	75	10	1.4026	0.0022	0.5081	5032	0.25
Siltstone	45	50	9	1.2623	0.0022	0.5081	4193	0.25
Siltstone/claystone	45	15	6	0.8415	0.0022	0.5081	839	0.30



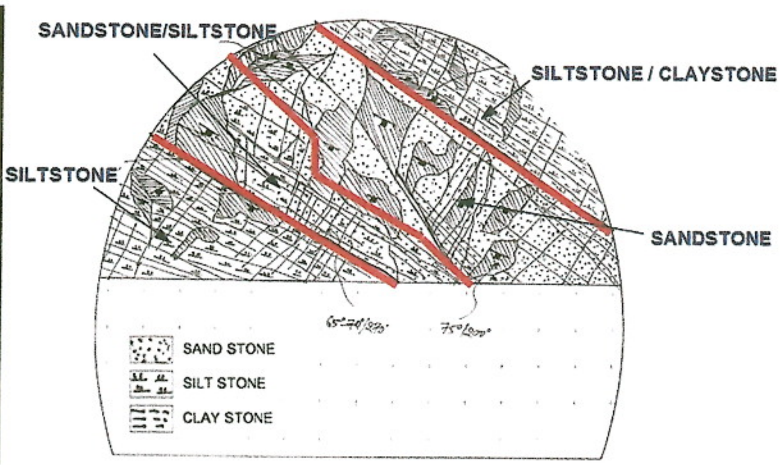
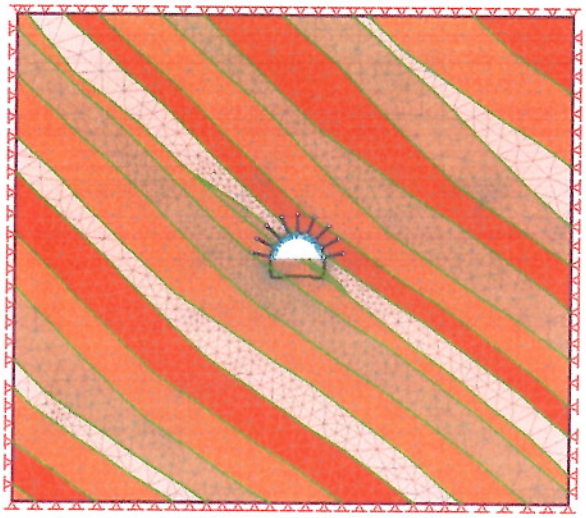


Fig. 4. Collapsed area of the right crown/side wall and geological face mapping at TM 820



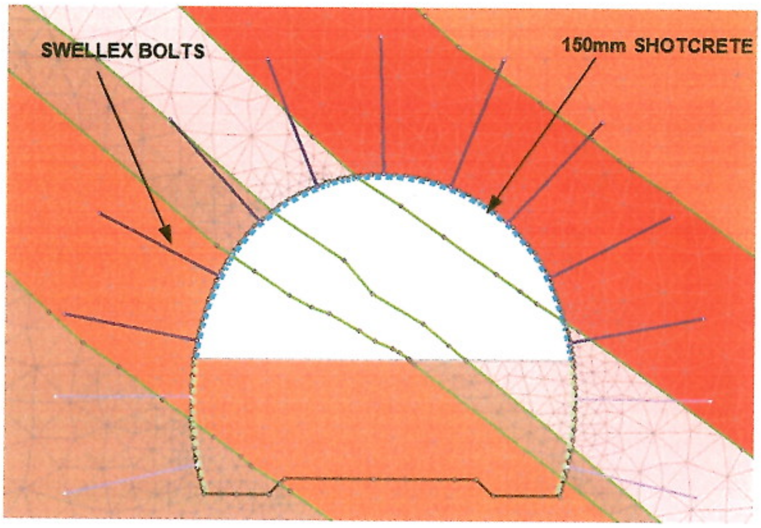


Fig. 5. Numerical model implemented in Phase 2

The achieved results in terms of displacements, calculated for the stage at which the primary lining is completed, were initially compared with the available monitoring data (notably the chord length reduction). The estimated unrecorded displacements occurring between excavation and the installation of targets, were obviously subtracted from the total displacements. The comparison (Fig. 6 top) proves the reliability of the numerical model and enables the highlighting of the critical zone where cracks were initially noticed in the shotcrete. The analysis moved forward with the comparison of the last available readings (a few hours before the instability phenomenon) with the maximum calculated radial displacement. In this respect, a potential creeping behaviour of the claystone/siltstone layer was identified, reasonably accentuated by the presence of water. The outcome proved very interesting, and the exact zone where the collapse had occurred was identified by the analysis, thus offering a reasonable explanation of the interaction between the real state of stress and the observed rock mass geomechanical and geostructural conditions.

The yielded elements (Fig. 6 bottom) are mainly concentrated in the weak siltstone/claystone layer on the right crown-side wall and the Swellex rock bolts installed at the right side wall are almost yielded. Rock bolts were still in place after the collapse due to the residual capacity at

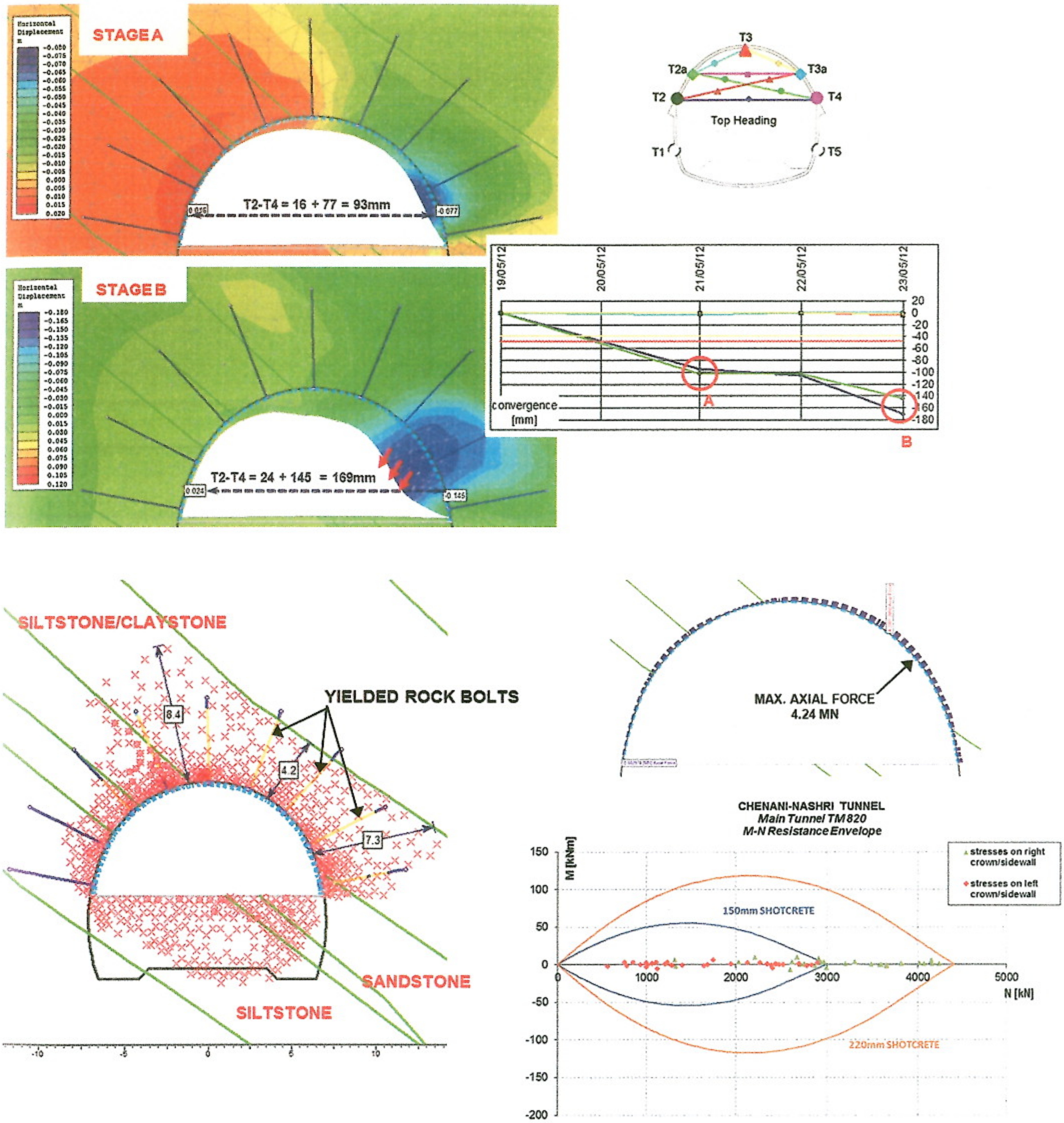


Fig. 6. Comparison between monitored readings and numerical analysis results (above). Yielded elements and N-M primary lining structural verification (below)

their ends, which was confirmed by the numerical analysis. The highest stresses in the shotcrete were recorded in the right crown/side wall, where the maximum compressive axial force in the lining is not compatible with the compressive strength of the 150 mm shotcrete, causing the development of cracks as seen on site. Consequently, a stronger support was required as already confirmed by the application of the "GDE Multiple Graph" ("B1" section with lattice girders and greater thickness of shotcrete, in place of the section type "B1*" applied in that sector; see Fig. 3).

4 The importance of the monitoring system (GDMS) during construction

GDE has specifically implemented for the Chenani-Nashri Tunnel a sophisticated 3D-monitoring system (GDMS), which is also easy to read and very useful for understanding the response to the excavation and checking the adequacy of the installed support. The following two-level action plan was proposed, with a set of actions to be performed should the convergence trigger limit be exceeded:

- Attention level: a percentage of the predicted deformation. On exceeding this limit, the frequency of readings

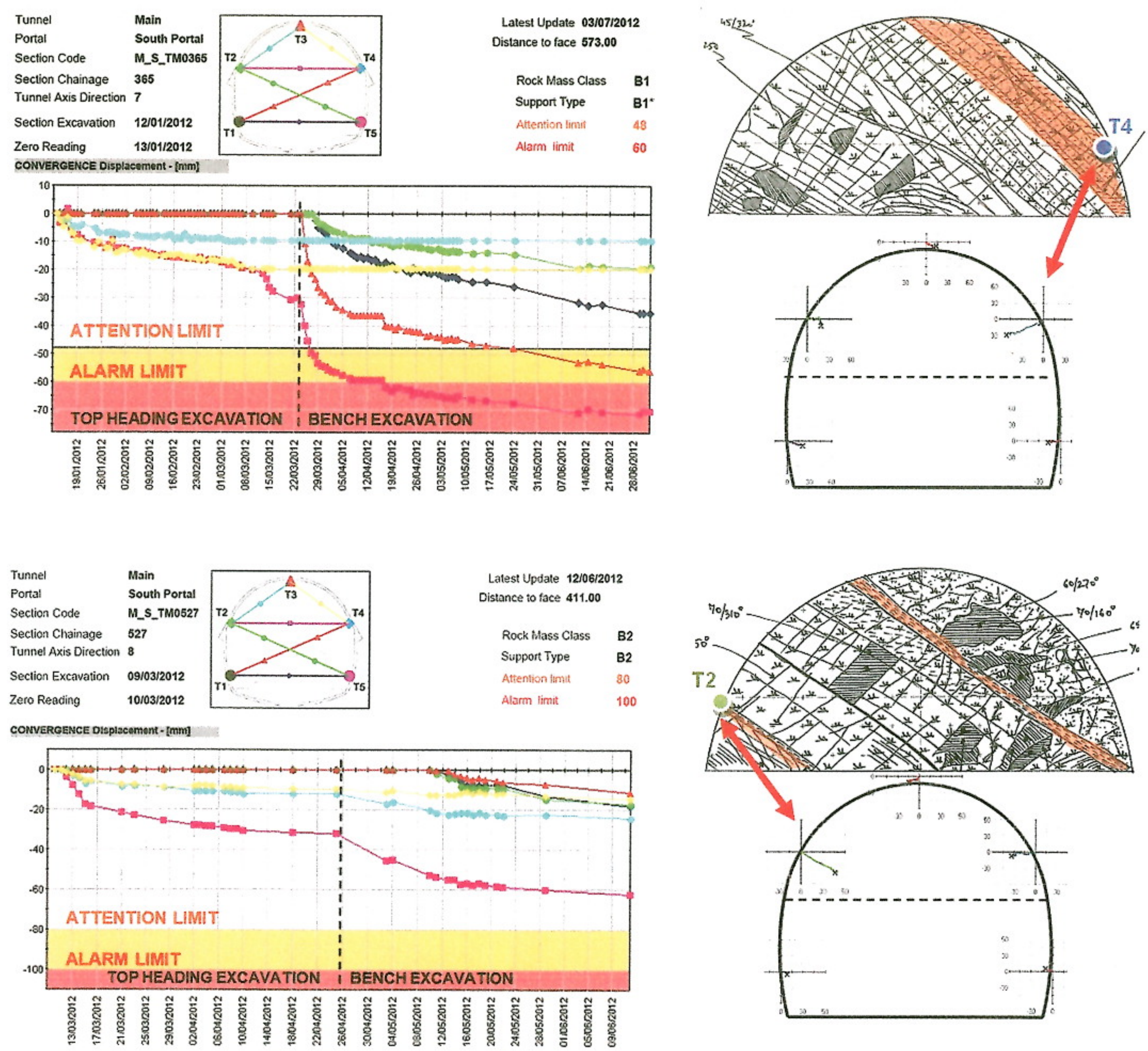


Fig. 7. Geological face mapping correlated with the monitoring results at TM 365 and TM 527

will be increased in order to evaluate the deformation speed and the convergence trends. This low limit allows enough time to assess convergence trends and to decide appropriate countermeasures should the movement continue to the alarm limit.

- Alarm level: the 100 % predicted deformation, as calculated at the design stage (equivalent to the ultimate support capacity of the section). Surpassing this limit triggers the procedure for action and countermeasures.

The example presented in Fig. 7 refers to the monitoring sections at TM 365 (support class "B1*", with 150 mm FR shotcrete and rock bolts) and at TM 528 (support class "B2", with 250 mm FR shotcrete, lattice girders and rock bolts).

The monitoring section at TM 365 showed a 20 mm chord displacement during top heading excavation (with the highest displacement at optical target T4), exactly corresponding to the weak inter-bedded siltstone and claystone layers. Displacements increased by over three times during bench excavation at the same location. By exceeding both attention and alarm limits, along with the recorded cracks in the shotcrete, the monitoring data highlighted the necessity to strengthen the primary support and apply a heavier support class, "B2". This proved to be a very effective countermeasure, ensuring the control of maximum displacement and bringing it down within the limits. As a matter of fact, with reference to monitoring section at TM 527, where B2 support class had been applied since excavation, deformations during the benching phase increased by "only" two times compared to those recorded during top heading excavation. In this case, a better response of the rock mass was reasonable due to the application of a stronger support as well as to the prevalence of intermingled sandstone and siltstone against thin layers of claystone.

Continuous and prompt assessment of the monitoring data allows selection of the appropriate support sys-

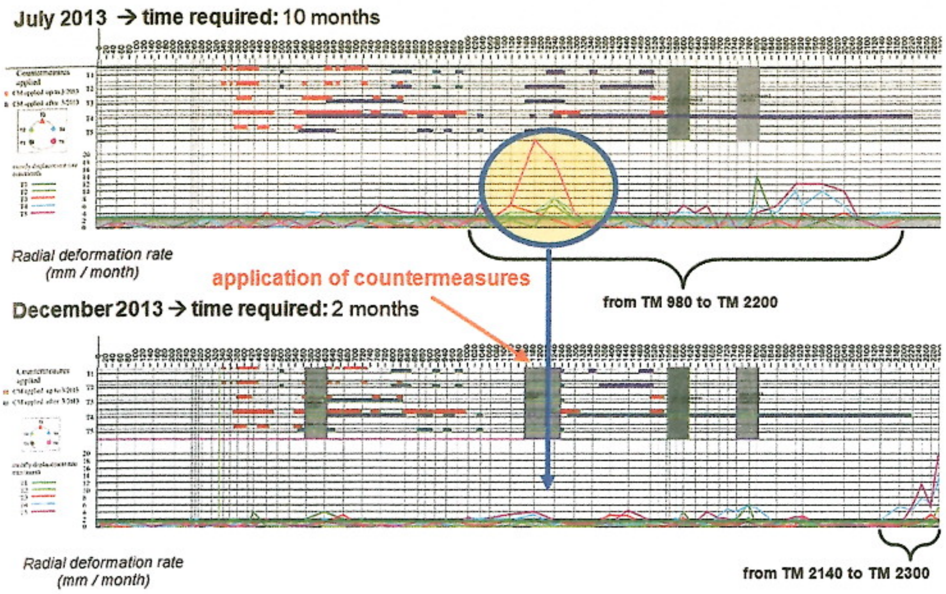


Fig. 8. Time frame for achieving a reduction of the radial deformation rate in the main tunnel, before (above) and after application of countermeasures (below)

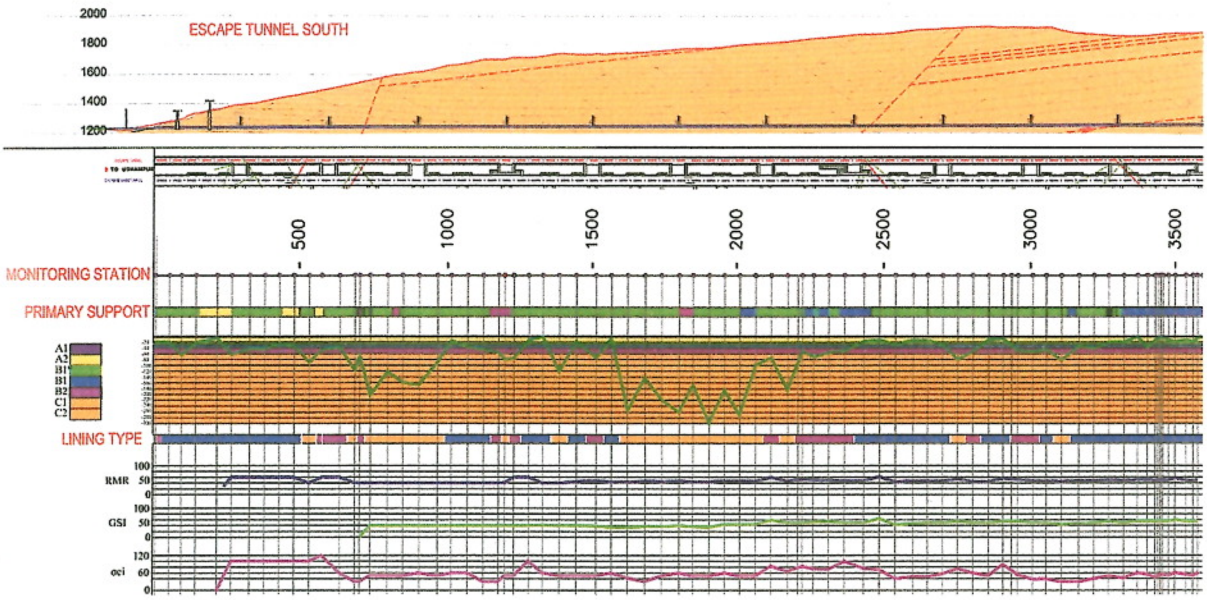


Fig. 9. Envelope of the convergences in the escape tunnel along with main geomechanical parameters

tem to be installed and possibly also the definition of the required countermeasures to limit the deformation, which may affect the overall stability of the tunnel. In addition, especially in an extremely complicated geological context as is the case in the Chenani-Nashri Tunnel, the design sometimes cannot cover all the potential or unexpected risks and local adjustment becomes inevitable. A proper monitoring system helps a lot in this sense: after the interpretation of monitoring results along with detailed back-analyses, it was possible to really understand the phenomenon and to manage it in an effective way. Fig. 8 shows the radial deformation rate (mm/month) related to the main tunnel section from the south portal at two different periods. In July 2013, deformations at TM 1200 were not yet fully stabilized ten months after face excavation, proving a marked time-dependant behaviour of the rock mass in that area and consequently an increase in the displacements.

Only after the installation of 9 m long self-drilling bolts at 1.5 m × 1.5 m spacing, which were able to "pack" the weak strata of siltstone-claystone, was it possible to stabilize the deformation. In December 2013, all deformation rates had been brought almost within 2 mm/months, so as to fulfil the restrictive criteria for placing the final lining mentioned in the contractual technical specs. This could have led to a delay of the lining casting works. This goal was finally achieved, mainly through the proper set-up of the support based on the monitoring results.

In Fig. 9, the envelope of the convergences along the escape tunnel is shown; high convergences (in some cases up to 300 mm) were registered up to TM 2400. In practice, the monitoring results confirmed the accuracy of the preliminary GDE assessment (based on the on-site geostructural surveys) performed during the early stage of the design (Fig. 10): a very strong anisotropic state of stress was

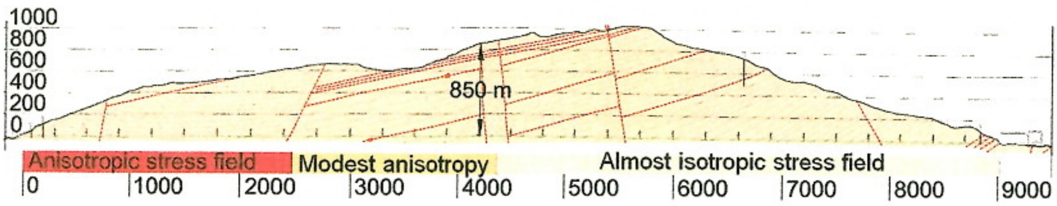


Fig. 10. Expected degrees of the anisotropic state of stress defined in the design stage

likely to be expected from south portal to about TM 2400, a modest anisotropy up to the tunnel stretch where the overburden reaches about 850 m (TM 4300), and an almost isotropic stress field for the remaining tunnel length until reaching the north portal. This means that the degree of anisotropy, which certainly depends on the different orientation of the strata along the tunnel, inevitably affects the response of the rock mass to excavation, as highlighted with the back analysis.

The monitoring system is currently widely applied for selecting the final lining (which has been designed for long-term loading) along with an "application criteria" defined by GDE to speed up the selection of the most appropriate class based on the deformations and on their allowable limits. As shown before, some critical stretches from the south portal have been identified during the excavation where a fully reinforced lining needs to be specified, also to avoid tunnel re-profiling, which typically results in time-consuming and extremely unsafe work. In these sectors, according to the registered deformations, big plastic zones are expected to act on the lining and back-analyses have been performed to check the structural capability of the lining itself with less thickness available. On the contrary, the monitoring system has enabled the identification of many tunnel sections, in which the lining will be partially or even completely unreinforced (e.g., at northern end, where geostructural conditions are better and the field stress is almost isotropic).

5 Conclusions

The behaviour of flysch formations cannot be easily determined due to the uncertainties in characterizing their anisotropy and heterogeneity. As demonstrated well by the results of the numerical analyses, the stability of the excavation in the flysch rock masses mainly depends on the prevalent expected lithology (sandstone, siltstone or claystone), on the geostructural conditions (favourable or unfavourable strata orientation) as well as the combined effect with the real state of stress. In this complicated medium, an appropriate monitoring system plays a fundamental role in highlighting potential risks and in selecting the most appropriate support system, and can thus help to optimize the overall excavation process and avoid time-consuming and costly remedial interventions.

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