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Airborne Electromagnetic Surveys for Carrying Out Feasibility Studies for Constructing Road and Rail Tunnels in Himalaya

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ABSTRACT

The Norwegian Geotechnical Institute (NGI) has been involved in several studies over the past two decades for constructing underground structures in the Himalaya. Recently, a detailed feasibility study was performed for a new road tunnel in the Bhutan Himalaya. The study included engineering geological mapping, rock mass characterization, geophysical investigations and numerical modelling for verifying the rock support requirements in the tunnel. Advanced airborne electromagnetic (AEM) surveys were performed along the tunnel corridor to provide information on the rock mass quality along the potential tunnel alignment and for visualizing the existing sub-surface geological conditions. Specifically, high resistivity areas i.e. competent bedrock was distinguished from low resistivity areas i.e., incompetent or weathered rock. The rock reinforcement requirements estimated from the Q-system of rock mass classification were verified through both finite element and distinct element modelling. This paper describes in detail the various studies performed along the proposed alignment for gaining an insight into the prevailing (in situ) rock mass conditions at the proposed site. In the Himalaya, which is generally characterized by steep slopes, lofty hills, and complex geological and tectonic settings, such studies are warranted for planning new and upcoming underground projects.

Keywords: Engineering geology; Geophysical methods; Rock mass characterization; Tunnelling

1. INTRODUCTION AND CASE STUDY

The construction of a new tunnel usually requires a detailed feasibility study. The purpose of such study is to plan and prepare for construction, including the support installation, the possible stability problems, and more. Not conducting these investigations may lead to significant delays and increased costs, as experienced by the numerous projects where large and unforeseen problems occurred during construction. Normally, the expenditures for engineering geological investigations and site characterization should be at least 3-5% of the estimated project cost. Recent advances in geoscience have however given us new tools to investigate the rock mass quality at different scales. The application of new tools with traditional approaches may help in a better planning of the tunnel construction. Practical examples indicate that small investments at the beginning of a project could save a lot later in the project (and in the longer term). This paper describes some of these methods, and in particular geophysical approaches and some numerical codes mainly taken from Bhasin et al. (2016). For each method, a theoretical description is proposed followed by a practical example and a few results from a selected case study.

The case study involves the planning of a new road tunnel in the Bhutan Himalaya between the capital, Thimpu, and Wangdue (Fig. 1). Both cities are fast growing areas in Bhutan and the existing 70 km long road between Thimpu and Wangdue is not well suited for this increased traffic. The road is steep, winding, narrow, and goes over the Dochu La pass at about 3100 m. In addition, there exists ice and snow in winter, and unstable road cuttings and slopes during the rainy season, which have caused repeated problems. The improvement of the old road is not considered relevant and a new road link was proposed. This new road would reduce the traveling distance by 36 km. It would, depending on the options, include either a 10.5 or 14.5km long tunnel. The construction of the tunnel, including the choice of the most favorable route and the design of the rock support, was studied using a broad range of methods, from geological surveys to geophysical investigations, including also numerical simulations and cost analyses.

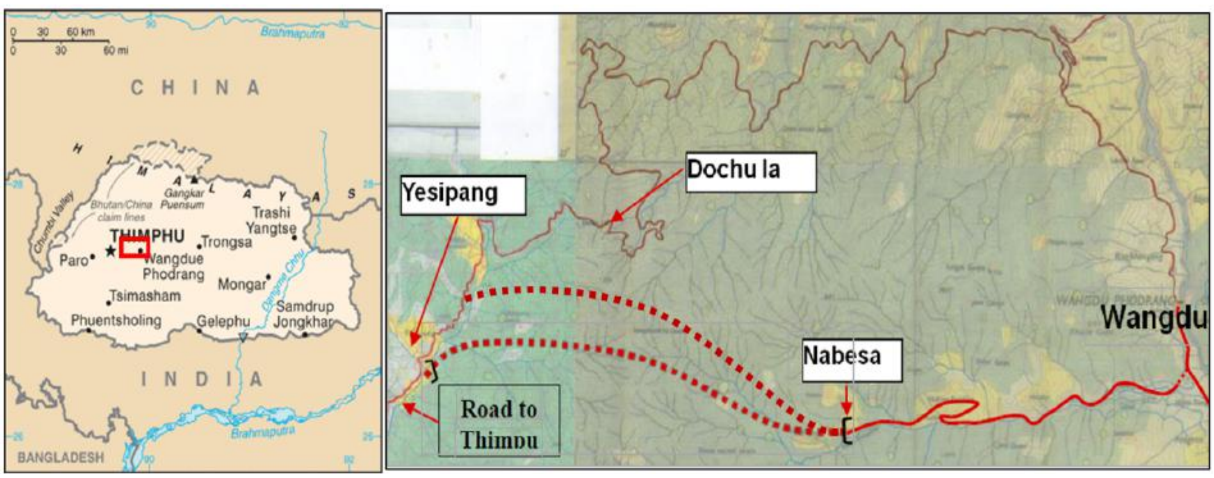

Figure 1: Location of the 70 km long old road over Dochu La pass (3100 m) and the two new road tunnel options (red doted lines) from Yesipang to Nabesa

2. GEOLOGICAL SURVEY

The first step of a feasibility study is the geological investigation of the area considered for the construction of a new road and/or tunnel. This includes studying the available geological maps, but also taking field surveys. Relevant information needs to be gathered, like the geology of the area, the presence of joints and weakness zones, and other local specificities (e.g. weathering depth, secondary mineralization in cracks etc.). When possible, the rock mass quality should also be assessed, for example using the Q-system (NGI, 2015). The Q-system was specifically developed from tunnel case histories in contrast to RMR which was based on mining case histories. Several correlations exist between Q and RMR values so there should not be a great difference in the classification of rock mass using either of the systems.

2.1 Geology

According to the geological map of Bhutan, the geology of the area between Thimphu and Wangdue is separated between an Orthogneiss unit (Cambrian-Ordovician) and a Lower metasedimentary unit (Neoproterozoic-Cambrian). The rock mass quality varies significantly along the tunnel alignment. The rock types are mainly gneiss, mica gneiss, meta-arkose and quartzite with varying content of mica and quartz. At some places the gneisses are veined. Lenses and layers of quartzite (common) and marble (less common) can be observed in the gneisses. The content of quartz varies a lot in the gneisses. The quartzites seem to vary from rather pure quartzite to meta-arkose containing lot of feldspar. The borders between gneiss and quartzite at many places appear as weakness zones.

2.2 Joints and Weakness Zones

Most of the rock is foliated, but varies from benching in massive rock, to rather schistose rock. The orientation of the foliation varies from place to place. Many joint directions have been observed and measured. The intensity of jointing varies from rather massive rock to very jointed sugar cube rock with few centimeters joint spacing. Joint and foliation orientations are not consistent along the tunnel alignment. No major fault zones or weakness zones could be observed in the geological survey, due to thick vegetation.

Deep weathering is observed at several places and particularly at the ridges and along the slopes. This weathering is also dominant along most of the road cuttings along the existing main road from Thimpu to Wangdue. The degree of weathering varies from completely decomposed rock, which is similar to soil, to partly weathered rock.

2.3 Rock Mass Quality

The rock mass quality has been mapped using the Q-system (Barton et al., 1974; NGI, 2015). The Q-system is a classification system for rock masses with respect to stability of underground openings. Based on estimation of six rock mass parameters, a Q-value for a rock mass can be calculated. This value gives a description of the rock mass quality. The different Q-values are related to different types of permanent support (NGI, 2015). The Q-system is developed for use in underground openings. However, the system can also be used for field mapping, during pre-investigation for tunnels and caverns. The reliability of the results of the field mapping will depend on the available rock outcrops. Evaluation of the Q-value may be possible with a reasonable degree of accuracy if the outcrops are vast and of good quality. The rock mass near the surface will often be more jointed than unweathered rock masses at a greater depth. If there are few outcrops, often only the competent rock masses will be visible. More jointed or weathered rock masses may be covered by soil.

Table 1: Preliminary estimation of the rock mass quality for both alternatives, given as a percentage of the total length of the tunnel

Q-values	> 10 Good	1-10 Poor or fair	0.1-1 Very poor	<0.1 Extremely poor
10.5 km tunnel	5%	58%	25%	12%
14.5 km tunnel	5%	65%	23%	7%

Q-values in the project, estimated during the field survey, vary from 0.01 (extremely poor) to 30 (good). A preliminary estimation of the rock mass quality was made depending on the alternative chosen (Table 1).

2.4 Limits to Field Investigations

Field investigations can have several sources of uncertainties and some practical limitations. The mapping in the project for example was mainly a spot check along the selected route, and covers only a minor part of the area along the tunnel alignment. The area is covered by dense forest, which makes observations of rock difficult. Rock exposures were observable only at some locations. Fresh rock was observed in the deep gullies and riverbeds. Probably the water has washed away the weathered cover. Consequently, the rock types, the structures and the rock mass quality can be observed mainly in the riverbeds and gullies, and in a few outcrops of rock. For this reason, other approaches were used to investigate the rock quality and plan the construction and the support of the new road tunnel.

3. AEM AND RESISTIVITY

3.1 Method and Principles

The AEM method is based on the physical effect of electromagnetic induction where an electrical current is induced in the ground and thus a secondary magnetic field is created. This secondary magnetic field is governed by the electrical resistivity of the ground. AEM systems measure the EM time decay or frequency response and the related resistivity distribution is subsequently obtained by inverse modelling. Time-domain systems (TEM) measure an EM step response decaying with time. They are generally well suited for deeper investigations due to the higher transmitter moment. Some TEM systems can provide highly accurate and well calibrated data.
AEM data provides a powerful tool for geotechnical projects due to coverage and survey speed. Significant cost reductions can be achieved by planning geotechnical drillings based on the preliminary geological model derived from AEM. Integrated with AEM, limited drilling sites can be linked and combined to a model covering the complete area of interest.

Time-domain systems (TEM) measure an EM step response decaying with time. They are therefore generally well suited for deeper investigations due to the higher transmitter moment. Frequency-domain systems (FEM) measure continuously at several frequencies. They tend to be superior for geology with high resistivity, maintaining high near- surface resolution. The choice of system depends primarily on (but is not limited to) the desired investigation depth and resolution, the terrain, and the resistivity contrast to be mapped.

AEM should be the first ground investigation step. Drilling locations can then be planned efficiently based upon AEM results. Subsequently drilling results should be incorporated in AEM data interpretation and visualization leading to a combined geological model (e.g. bedrock topography). AEM is better suited for regional-scale projects rather than isolated projects because costs are relatively high for small surveys. Parallel flight lines covering an area are preferable over flights which are aligned parallel to infrastructure long, linear infrastructure. Survey extent is limited by the presence of power lines and urban infrastructure.

3.2 Experience from Case Study in Bhutan

NGI's Geophysical subcontractor, Danish airborne electromagnetic (AEM) provider SkyTEM, carried out an AEM survey covering a tunnel corridor between Thimpu and Wangdue in Bhutan. A total of 158.1 km flight lines were flown. The high altitude in Bhutan, the rugged topography and the gusty winds in the survey area were a challenge for the survey. The nominal terrain clearance for the towed system is usually 30 m, but is larger over forests, power lines, or any other obstacles. In the present survey the instrument altitude varied considerably, between 30 and 400 m, with a mean around 110 m. The data quality is therefore quite variable within the survey area. The bin spacing (horizontal distance between data points) is approximately 36 m.

3.3 Results and Interpretation

The survey target is the resistivity contrast between the weathered layer, the underlying intact bedrock, and possible weakness zones in the bedrock. High resistivity areas (competent bedrock) can be distinguished from low resistivity areas (incompetent and/or weathered rock). The regional resistivity is quite high (mostly above 1000 Ω.m), which is typical for gneisses. Due to this resistive background, the AEM depth of investigation was higher than anticipated, mostly between 300 and 800 m.

Two resistivity maps are presented because of their geological interest. Figure 2 presents the horizontal mean resistivity map from the surface to 25 m depth, and Fig. 3 between 300 and 350 m depth. The green to blue colours (high resistivity) indicate resistive material typical for intact bedrock whereas red to yellow (low resistivity) indicate conductive material that could be weathered rocks or weakness zones.

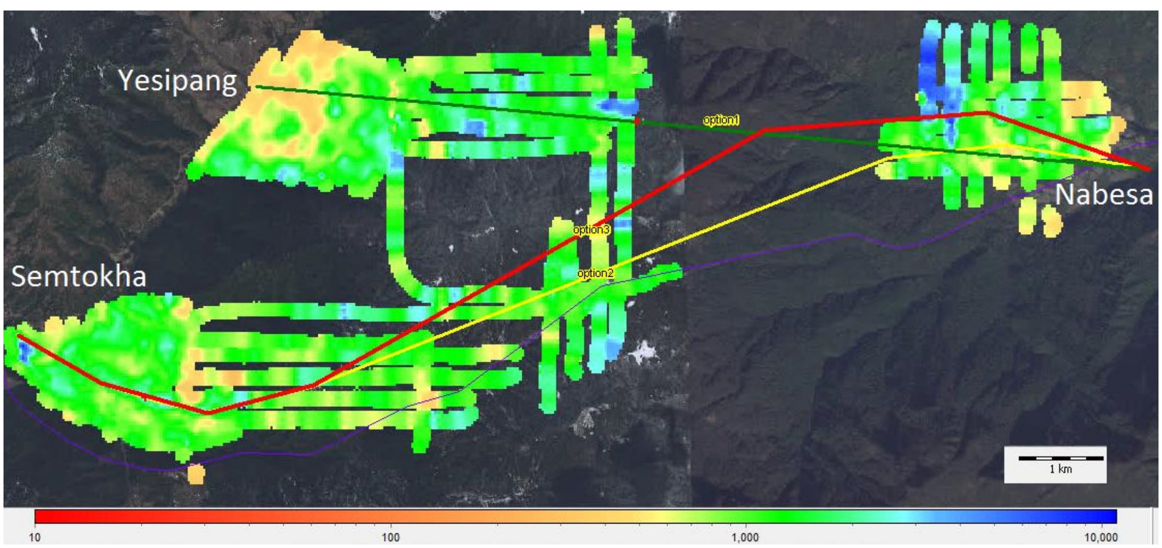

Figure 2: Average resistivity in a layer from 0 to 25 m depth below surface, for the entire survey area. The scale is in Ωm: red is conductive while blue is resistive

Usually, and as expected, near surface resistivity are lower in the valleys than at the peaks. The highest surface resistivity is found near steep cliffs. In average, the near surface resistivity is quite high (above 500 Ω.m). A deeper region of lower resistivity (orange colour) is observed 1.5 to 3 km east of Yesipang (Fig. 3). A region of very low resistivity (red colour) is also observed in the south west of Nabesa (300-350 m deep, see Fig. 3). This conductive anomaly could correspond to a weakness zone.

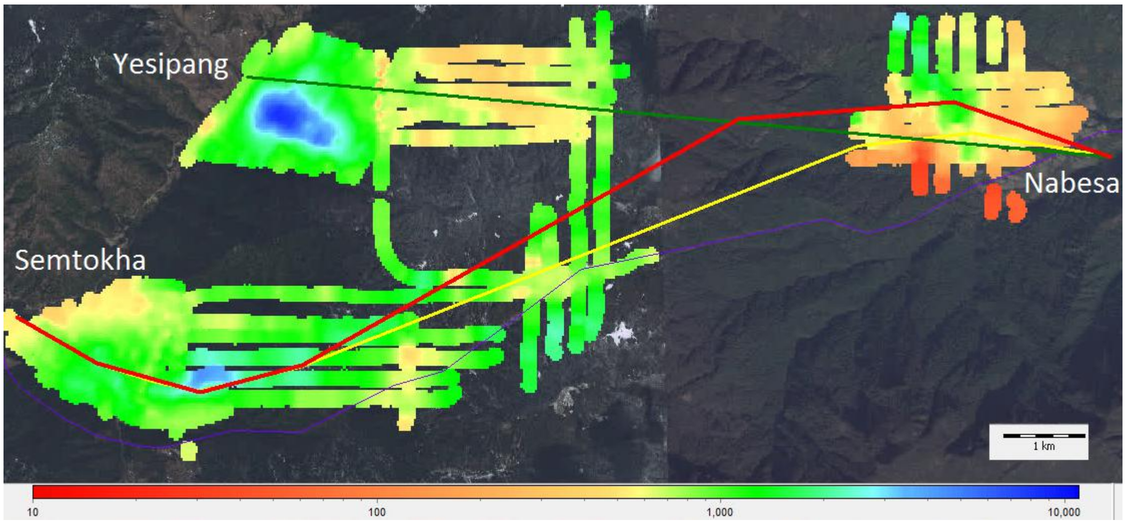

Figure 3: Average resistivity in a layer from 300 to 350 m depth below surface, for the entire survey area. The scale is in Ωm: red is conductive while blue is resistive

More specifically, a deep conductive zone, south west of Nabesa portal, may intersect the initial alignment for the tunnel (also see next section). The assumption of a potential weakness zones needs to be confirmed by deep boreholes. The conductive anomaly is however relative shallow (approx. 110 m) in some parts of the survey area and several locations for future borehole investigation were proposed based on these results.

The western portals are less critical and no strong conductive anomaly are observed in these two regions. The low near surface resistivity at Yesipang portal indicates that the bedrock is altered near the road along the valley but the resistivity increases rapidly with depth (approximately at 25-50 m depth) and also towards the east. The near surface resistivity at Semtokha is higher than at Yesipang and therefore the bedrock may be less altered. However it decreases with depth (approximately at 50 m depth). These results led to recommend careful planning in these area.

3.4 Weakness Zone

The deep conductive anomaly located south west of Nabesa is a concern and five vertical sections in the area were extracted to better visualize this potential weakness zone. A conductive anomaly is indeed observed along the five vertical sections (see an example in Fig. 4). Results may indicate that this anomaly deepens towards the North, as it would be expected for Himalayan nappes. However, the AEM accuracy is limited at such depth. Neither the exact resistivity profile nor the exact shape (dip angle and thickness) can be retrieved. The preliminary geological investigation had suggested that the valley that runs SW-NE just south of the Nabesa portal could indicate a weakness zone. The conductive body is most likely the same rock type as the one that was eroded by the river in the valley. The roof of the conductive anomaly is located at an elevation comprised between 1500 and 2000 m (Fig. 5).

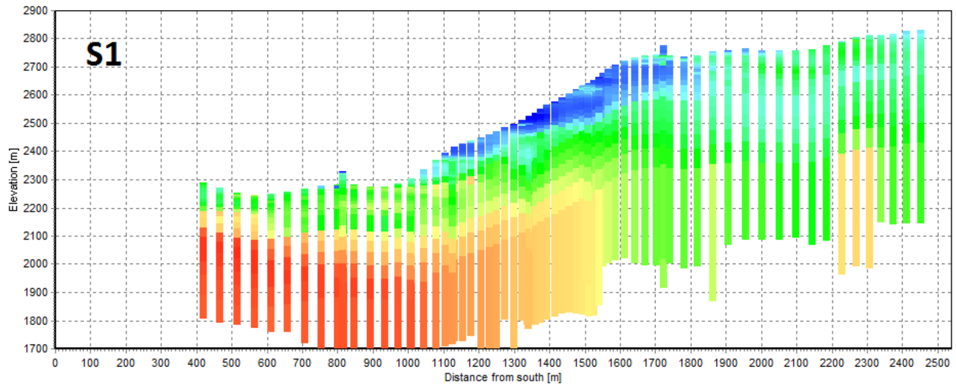

Figure 4: Resistivity in a vertical section (South-North) West of Nabesa. The same scale as in Fig. 2 is used: red is conductive while blue is resistive

3.5 Limitations of AEM method

It is important to remember that AEM technology is complimentary to ground surveys and not a replacement for physical sampling of the ground. AEM survey saves time and cost through reduction of drillings required on the ground. There are some limitations of the method which are highlighted underneath:

a) Distinguishing between good and exceptionally good rock mass quality
b) Resolving small features such as joint sets in hard rock
c) Sediment thicknesses of less than 5 m
d) Distinguishing between soil and bedrock that have the same resistivity, e.g. thickness of clay over shale bedrock

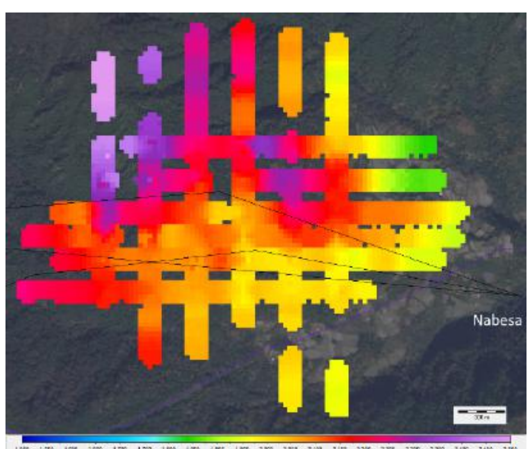

Figure 5: Elevation of the potential weakness zone close to the Nabesa portal

4. REFRACTION SEISMIC SURVEY

4.1 Method

The sub-surface shear wave velocities (1D) were mapped using the multi-channel analysis of surface waves (MASW) technique as well as P-wave velocities through P-wave diving wave tomography. The former technique is based on the geometrical dispersive nature of surface waves (here, Rayleigh waves), meaning that their phase velocity changes as a function of frequency (Yilmaz, 2001). The latter technique assumes spatial velocity gradients causing wave diving, of which the first arrival times can be used to infer sub-surface structure and P-wave velocity information (Park et al., 1999b; Socco and Strobbia, 2004).

Data were collected at a single location, close to one of the tunnel portals at Yesipang. A total of 24 vertical geophones (4.5 Hz corner frequency) were used. Both source and receivers were moved for successive recording sequences. Typically, the spacing between neighboring geophones upon data collection was 5 m, with shot points spaced roughly every 10 m apart. For each source-receiver configuration, the shots were repeated in order to allow (i) selecting the highest-quality traces for analysis and (ii) improve signal-to-noise ratio through stacking.

Data were recorded with a sampling interval of 0.5 ms (2 kHz, implying 1000 Hz Nyquist frequency) over 1.2 s, with 0.2 s pre-trig time. The seismic source was either a 6 kg sledgehammer, impacting on wood, or a specially-designed weight drop (60 kg) that was manually released from approximately 1 m high and guided by a vertical rod, impacting a rubber slab on the ground. As a trigger, a geophone deployed in the immediate vicinity of the impact location was used. For this site, with maximum offsets limited to around 140 m, penetration does not exceed approximately 35 to 45 m.

4.2 Results

Result of the refraction tomography is obtained for various models and conditions on the velocities (see an example in Fig. 6). The difference in the models relates to constraints on the P-wave velocities and gradients, rather than the resolution or other parameters in the inversion. For all models, the penetration is down to about 35 m at the center of the receiver array, and less to either side. The transition to bedrock lies probably around 20 to 25 m at this site.

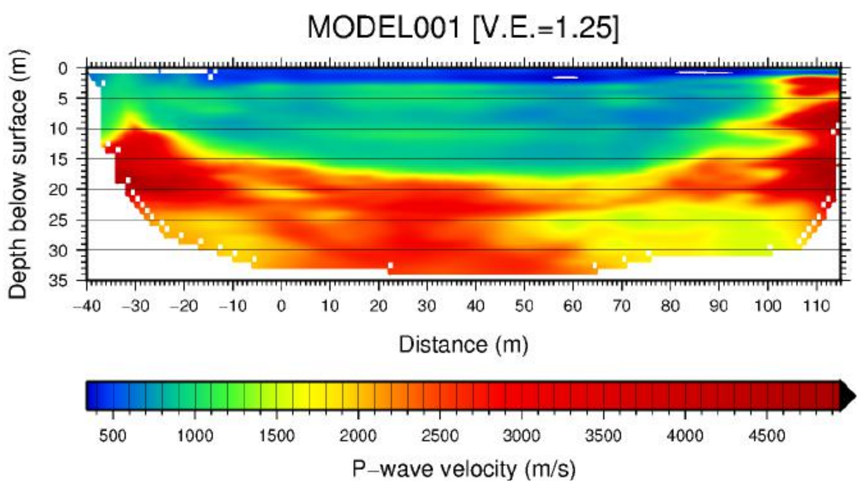

Figure 6: Example of result from diving P-wave tomography from the 17 interleaved shot gathers. The transition to bedrock occurs around 25 m below surface.

The data were also inverted for shear wave velocity and layer thickness using the WinMASW inversion software. A significant increase in S-wave velocities appears at about 25 m depth, which corresponds well with the results from the P-wave tomography (Fig. 6). For the same area the results from ground refraction seismic agree quite well with the results from airborne AEM survey which also indicated high resistive areas (competent rock) below 24 m depth.

Figure 8 shows the AEM results with a vertical cross-section indicating the depth of bedrock at around 25 m depth near the Yesipang portal.

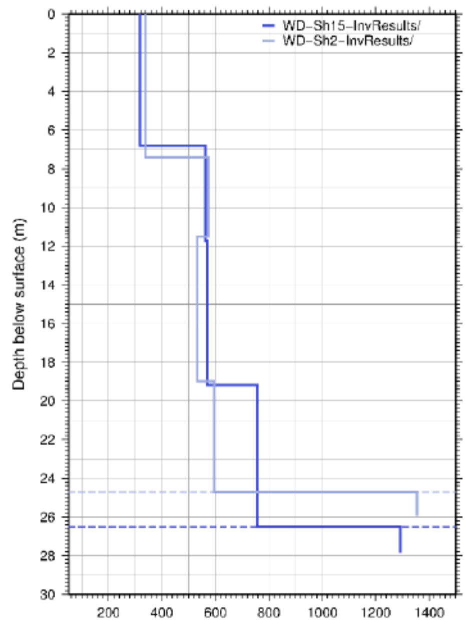

Figure 7: Results of fundamental surface wave inversion at Yesipang, from the interleaved shot gathers from WinMASW using shot positions 2 and 15, respectively. Results show the depth of the weathering zone. Un-weathered rock is expected to be found approximately 27 m below the surface

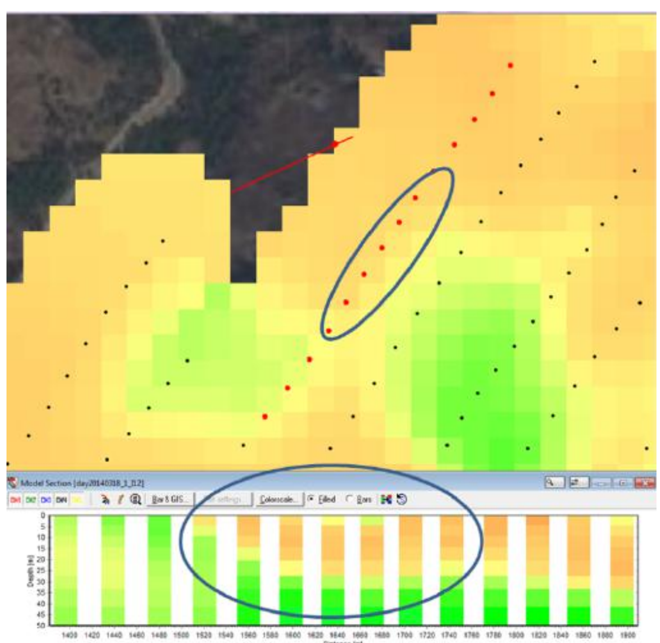

Figure 8: AEM results indicating high resistive areas at around 24 m depth near the portal area of Yesipang

5. NUMERICAL SIMULATIONS

Numerical simulations are useful for predicting the behaviour of underground excavations in both weak and jointed rock mass conditions. NGI has successfully predicted the behaviour of rock masses in underground structures in the Himalaya using numerical techniques (Bhasin and Pabst 2013 and Bhasin et al, 2008). Both static and dynamic (earthquake) analysis have been performed using various codes.

5.1 Q-system

Rock support for the new road tunnel was estimated using the Q-system, based on the Q-values obtained from geological and geophysical surveys:

- Q > 10 : spot bolting 2.5/m, shotcrete 6 cm (1 m^3/m);
- Q = [1 ; 10] : systematic bolting 3.5/m, shotcrete 8 cm (2 m^3/m);
- Q = [0.1 ; 1] : systematic bolting 6.5/m, shotcrete 12 cm (4 m^3/m);
- Q < 0.1 : systematic bolting 12/m, shotcrete 20 cm (6.5 m^3/m), 0.5 RRS/m.

5.2 Numerical simulations

A numerical assessment of the rock support for the Thimpu-Wangdue road tunnel was carried out using a finite element approach with Phase^2 (v8, RocScience; 9) and a distinct element approach with UDEC (v6, Itasca; 10). These complementary analyses were used in order to catch the possible trends and behaviours, whether the main driving force was the rock mass deformation or the displacements along the joints (see Figs. 9 & 10). Rock support was simulated as prescribed by Q-system. A dynamic and a pseudo-static analysis were also conducted to assess the behavior of the tunnel while subjected to earthquakes. Typical and characteristic cross sections both in terms of stresses and rock mass quality were chosen for the analysis. The portal areas require usually a very specific assessment and were not considered in this study.

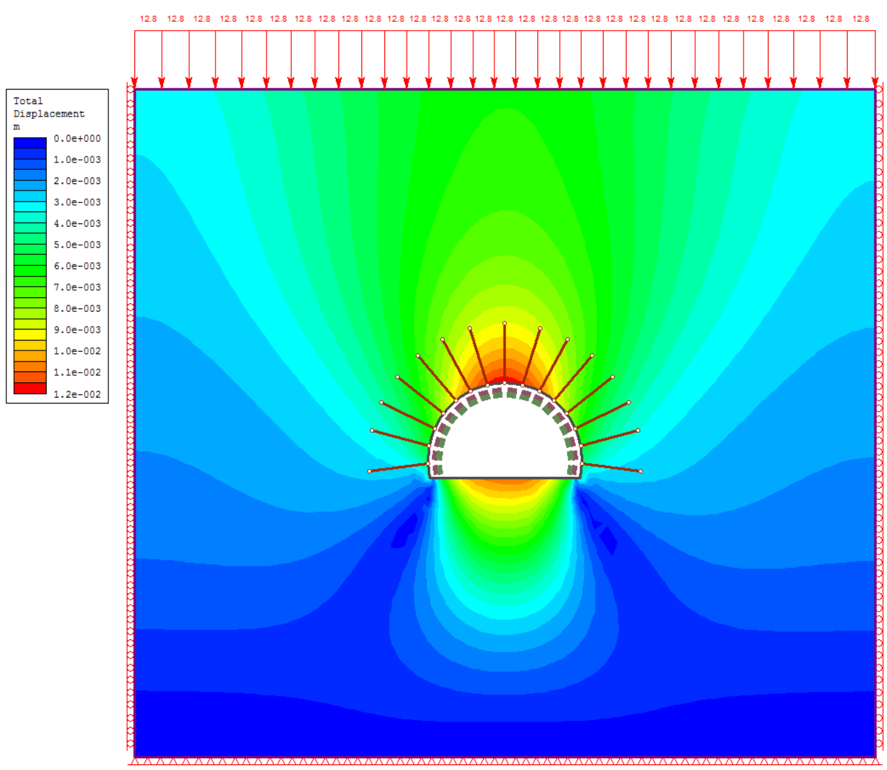

Figure 9: Example of results obtained with Phase^2 models. Total displacements are shown

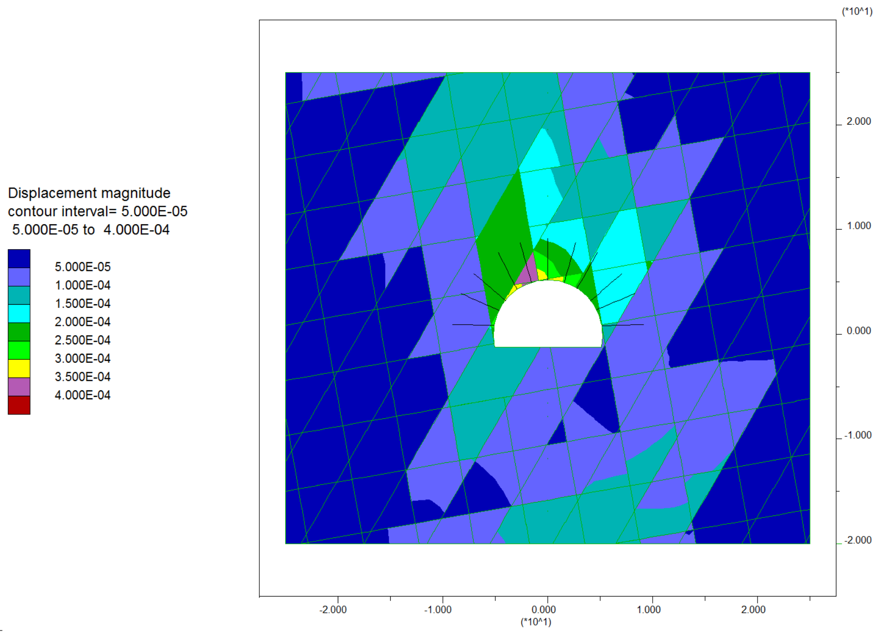

Figure 10: Example of results obtained with UDEC models. Total displacements are shown

The effect of the rock mass quality and the overburden (i.e. stresses) were assessed using the Phase^2 software (Fig. 11). It appears from the numerical simulations that the total maximum displacements (usually in the roof of the tunnel) increase, as expected, with the overburden and for poorer rock mass quality. The displacements for an extremely poor rock significantly increase for an overburden equal of larger than 1000 m, in direct accordance with support failure (Fig. 12). While the proposed support system remains intact for Q-values higher than 4, independently of the overburden, it partially fails for Q-values lower than 0.4 and overburden higher than 1000 m. The (shotcrete) liner seems to be more affected than the bolts. Simulations show therefore that the proposed support may not be suitable for the highest stresses, and that construction costs may increase in the case very poor of extremely poor rock is encountered at high depth.

An earthquake (with characteristics typical for these regions) would have, according to these models, little effect on the total displacements (Fig. 11) and would only slightly increase the support failure (bolts and liner).

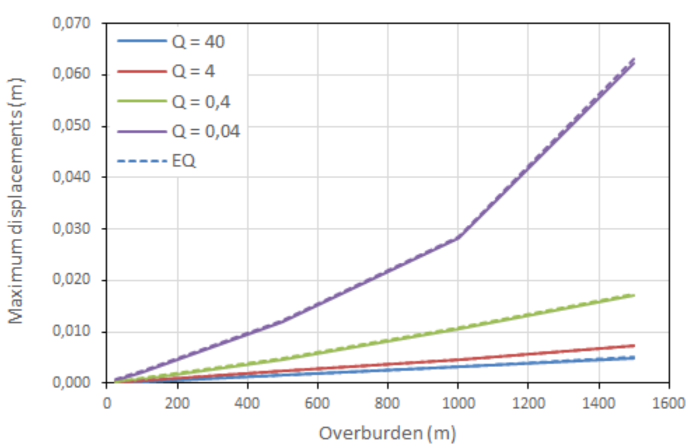

Figure 11: Simulated maximum total displacements (in m) for various Q-values and overburden. The effect of an earthquake is shown with dashed lines (see details in text)

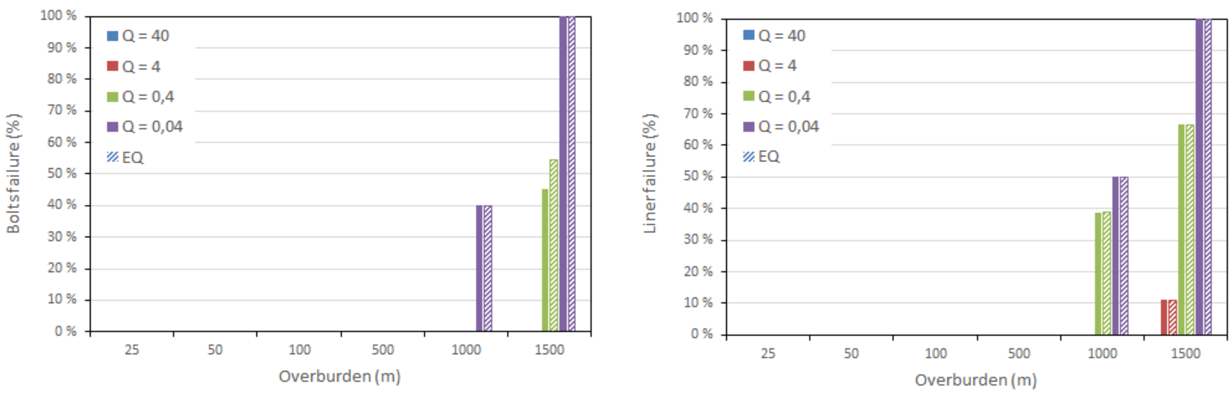

Figure 12: Bolts and shotcrete liner failure (in %) for various Q-values and overburden, under static and pseudo-static (earthquake or EQ) loadings

The results obtained with UDEC simulations (Fig. 10) are somewhat similar (at least regarding trends) to those calculated with Phase^2, but slightly differ due to the simulation approach (discrete elements vs continuous model). The differences in displacements between continuum and discontinuum models are attributed to the presence of joints in the discrete models. The spacing and persistence of joints have been exaggerated to simplify the discrete model. The convergence and the roof displacements increase with the overburden and for reduced Q-values (Fig. 13). Displacements for Q-values higher than 4 are negligible while it can exceed 1 cm or 4 cm for 1500 m overburden and Q-values of respectively 0.4 and 0.04. These values are similar to those obtained with Phase^2.

Support failure appears however somehow increased compared to results from Phase^2 (Fig. 14). While support for Q-values higher than 4 still remains intact for all overburdens, instabilities start appearing for Q-values lower than 0,4 for overburden exceeding 500 m. The bolts seem also more affected in these models, which can be explained by the discrete element approach (displacements occur along joints, thus putting the bolts under higher stresses compared to a continuous approach like Phase^2). The conclusion remains however the same, which is that the proposed rock support may be insufficient for the lowest rock qualities and the deepest parts of the tunnel, thus leading to a possible increase of the costs for rock support.

Earthquakes in UDEC are simulated in a much more realistic way than in Phase^2 (dynamic loading vs. pseudo-static loading). In UDEC, the earthquake is simulated by applying a sinusoidal shear wave to the base of the model. Its frequency was here chosen equal to 5 Hz and the signal applied for 3 seconds that is a total of 15 cycles of motion. Yet the effect of an earthquake with a PGA of 0.24 on displacements in the tunnel is limited when the Q-value of the rock mass is higher than 4. For Q-values lower than 0.4, convergence and roof displacements can however both be doubled (Fig.13). The support appears also to be significantly more affected than in Phase^2 models (Fig. 14). Even for the highest rock qualities, both the failure of bolts and liner is increased under a dynamic solicitation. The increase in failure remains however relatively limited and it is not believed that an earthquake would compromise the rock support more than it is under static conditions. The simulations carried out with UDEC also show that both the displacements and the rock support are more affected by an earthquake for small overburden (25 m) than slightly deeper (50-100 m) tunnels. It may indicate that a particular attention needs to be paid for earthquake preventive measures around the portal.

Results from the numerical simulations indicate that the recommended support, based on the Q-system, should be efficient to prevent instabilities in the case the rock quality is poor or better. It also shows however that for very poor and extremely poor rock qualities, rock support may need to be reinforced when the overburden exceeds approximately 1000 m. The effect of an earthquake with a PGA of 0.24 g (realistic for this region) is limited in most cases. There is an increase in displacements and support failure, but the situation does not significantly differ from static loading. Despite some slight discrepancies, the simulations with Phase^2 and UDEC lead to the same conclusions.

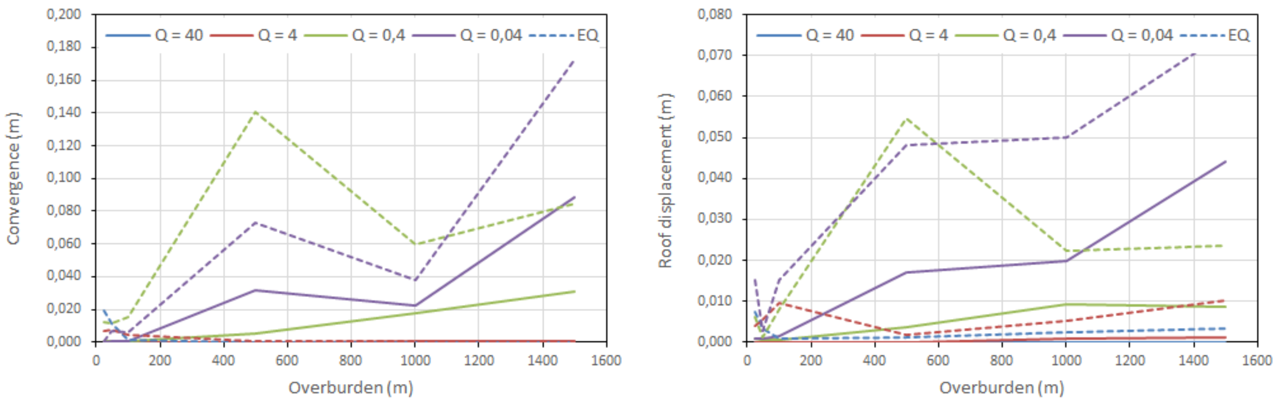

Figure 13: Simulated convergence and vertical (roof) displacement (in m) for various Q-values and overburden. The effect of an earthquake is shown with dashed lines (see details in text). Curves are not linear because of the numerical approach (discrete element method)

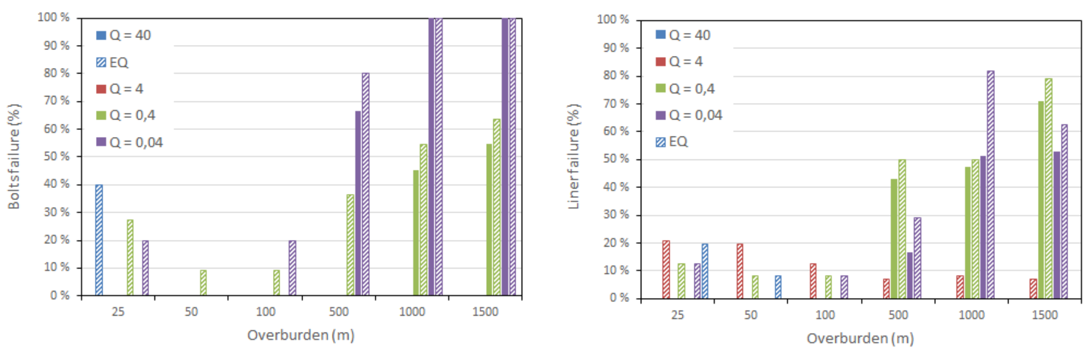

Figure 14: Bolts and liner failure (in %) for various Q-values and overburden, under static and dynamic (earthquake or EQ) loadings

6. CONCLUSION

The Norwegian Geotechnical Institute (NGI) has been involved in several studies over the past two decades for constructing underground structures in the Himalaya and has developed to this respect special competence in assessing rock mass quality using the latest geosciences advances. Recently, a detailed feasibility study was performed for a new road tunnel in the Bhutan Himalaya. The study included engineering geological mapping, rock mass characterization, geophysical investigations and numerical modelling for verifying the rock support requirements in the tunnel. Advanced airborne electromagnetic AEM surveys were performed along the tunnel corridor to provide information on the rock mass quality along the potential tunnel alignment and for visualizing of the existing sub-surface geological conditions. Specifically, high resistivity areas i.e. competent bedrock was distinguished from low resistivity areas i.e. incompetent or weathered rock. The rock reinforcement requirements estimated from the Q-system of rock mass classification were verified through both finite element and distinct element modelling. This paper illustrated the various studies performed along the proposed alignment for gaining an insight into the prevailing rock mass conditions at the proposed site. In the Himalaya, which is generally characterized by steep slopes, lofty hills, and complex geological and tectonic settings, such studies are warranted for planning new and upcoming underground projects. It is believed that the combination of the different approaches, which have been described in this paper, may help in a better planning of the tunnel construction in the Himalaya leading to significant cost savings in the long-term.

Acknowledgements

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