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Perceiving Geological Risk Using Tunnel Seismic Prediction During Tunnelling In Weak Sedimentary Rocks

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ABSTRACT

While tunnelling through weak rock mass, it is important to perceive the geological ground conditions in advance for timely planning of construction countermeasures. Various geological hazards such as face collapse, water-inrush and squeezing are commonly encountered in weak sedimentary rocks.

Tunnel Seismic Prediction (TSP) is a well proven technology delivering seismic parameters of P- and S-wave velocity and further relevant rock parameters such as dynamic Young's modulus and Poisson's ratio. In massive rocks of low porosity, the static and dynamic values of the Young's modulus lie close to each other, while for heavily fractured and clay bearing zones, large differences between static and dynamic parameters exist. Here, rock mass characterisation of Q or RMR usually correlate better with static modulii. However, it is seen from TSP data, that even dynamic modulii relatively correlate well with rock descriptions. The predicted rock mass classification and geological risks are validated through comparison against actual geological face logs during tunnelling.

Keywords: Tunnel seismic prediction; Weak sedimentary rocks; Geological risk; Rock mass classification; Seismic parameters.

1. INTRODUCTION

India is one of the fastest growing markets for tunnel construction over the coming years. The country has major needs for tunnelled infrastructure to improve its vast hydropower potential in Himalayas, and to improve road and railway connections across the country.

The construction of these structures is a challenge to engineering skill, when these are located in the weak and fragile rocks of Himalayan region. Himalaya is a young mountain chain and tectonically active. The rocks of the Himalaya, as mostly experienced through tunnelling, are incompetent, quasi-elastic with a number of folds, fault and thrust of various magnitudes and heavily charged with sub-terrain water.

Tunnelling at great depth through such a terrain with mixed lithology, varying tectonic behaviour and trapped water reservoir at considerable head becomes a very costly affair and hazardous operation due to the encounter of problems like squeezing, swelling, running ground, sudden ingress of water combined with a poor state of rock, excessive temperature and gases in rocks.

Almost all the major tunnelling projects in the Lower Himalaya have been beset with widely ranged construction problems. The most common sedimentary rocks are sandstone, shale, conglomerate, limestone and dolomite. Sedimentary rock can be very diverse in its stratification, i.e., the bedding can vary from limestone to shale in the adjacent bed. These abrupt changes in the rock can create tunnelling problems. The effect of stress and advanced weathering and weakening by the action of water can rise to problem especially where such rock type contain clay minerals.

Different methodologies are required to adopt while excavating very poor rock mass (Class-V) and exceptionally poor (beyond Class-V) rock mass. Ongoing investigation ahead of tunnel face during construction is important by using tunnel seismic that can help to define logistics before they are being required.

The study area is located in the Sub Himalaya in the Miocene Middle - Lower Shiwalik formation. The main geological formations are recent quaternary sediment and tertiary to pre-tertiary rocks. The tertiary rocks comprise the Subathu, Dagshai, Kasaulis and Shiwalik formation and consist mostly of sandstone, siltstone, shale & clay.

2. RISK ASSOCIATED DURING TUNNELLING IN WEAK SEDIMENTARY ROCK

The major problem in tunnelling is ground behaving differently than anticipated. Sedimentary rocks can be soft, unconsolidated and easily disintegrable. The Upper Siwalik contains thick beds of conglomerates and they are loose and fragile. Similarly, Lower Siwalik and Middle Siwalik have problem from alternating beds of mudstones and sandstone. In such alternating bands, mudstone can flow when saturated with water, which results overhanging sandstone beds. Such overhang jointed sandstone beds easily disintegrate into blocks. Some common types of geological issue have found from past tunnel experienced in weak rock as described below.

2.1 Shear and Fractured Zone

Shear zones are characterized by highly deformed, sheared, water charged, poor rock mass conditions. Serious tunnelling problems have been experienced when the rock mass is affected by multiple shear zones. The problems are mostly related to loose fall, chimney formation, squeezing/heaving and collapses due to less standup time of rock mass of Class-VI and beyond Class-VI ($Q=0.01$, $RMR<20$).

2.2 Ground Water Condition

Tunnelling through rock mass which is highly charged with ground water faces major problems such as:

(i) Heavy ingress of water in tunnel hampers the construction activities.

(ii) The saturated rock mass looses its strength as the shear strength (cohesion and friction) gets reduced due to lubrication, and failure of rock mass occurs from the crown and above spring level.

Investigation of groundwater condition is very important for successful completion of tunnel work. Though it is difficult to judge the groundwater condition during investigation stages, efforts are being made to probe simultaneously while advancing in water charged zones by probe drilling and Tunnel Seismic Prediction.

2.3 Folded Rock Sequence

In Himalaya, rock sequences are folded and refolded, regionally and locally, due to polyphase deformations. The rocks are tightly folded due to high compression in the close vicinity of major tectonic features like thrusts. The rock mass, present at the closures of synclinal and anticlinal structure behave differently when tunnels are driven through them. In anticlines the rock mass is highly fragmented/jointed at the closure and structurally controlled failures are anticipated at the crown of the tunnels, whereas in the synclinal troughs the rock mass is mostly water charged and at times act as huge water aquifers. The fold axes are usually traversed by numerous shear zones and present difficult ground conditions for tunnelling due to extremely poor rock mass. These problems severely affect the tunnelling schedule.

2.4 Presence of Low Cover Zone

As per past experience inedequate vertical and lateral cover surrounding tunnel can creat a serious problem during tunnel construction. It is significant to assess the geological condition of tunnelling media accurately in low cover zone so that necessary arrangment can be made to tackle the problem.

2.5 High Stress Condition

The rock mass under very huge cover (H) is sbjected to huge stress and during excavation the ambient stress condition of the rock mass get disturbed and rock tends to deform by releasing some of stress through squeezing in case of soft rock mass. Due to this, tunnelling work may stop for several months ($H>350Q^{1/3}$, and $J_r/J_a<0.5$; where Q is Barton's rock mass quality and J_r & J_a are Barton's parameters).

3. RISK MITIGATION USING TUNNEL SEISMIC PREDICTION

The rock quality is mainly related to rock mass strength, deformability, weathering and the presence of discontinuities. All these properties are somehow linked to each other as the rock strength may be influenced by discontinuities and foliation, and their orientation. During the planning phase, the designation of rock quality is mostly based on surface observations and borehole data. They are not as reliable as observations in the tunnel during construction.

Even in the tunnel, rock mass strength is difficult to estimate, because the strength and deformation of rock mass and an intact rock specimen differs. However, due to a certain scale effect there have been derived empirical formulae estimating the rock mass strength, such as Rock Mass Rating (RMR) (Bieniawaski, 1993), Geological Strength Index (GSI) (Hoek et al., 1998) and Q-value correlation (Barton, 2002).

Due to geological assessment constraints, non-destructive geophysical site investigations while tunnelling have developed and improved significantly over recent times. In particular, when site investigations from the surface are limited given the topography, tunnel seismic can detect lithological heterogeneities within distances up to hundreds of meters ahead of the face, many times more that of probe drilling alone.

Especially, tunnel excavations using tunnel boring machines (TBM) do not provide geological data of the tunnel face, and they often use continuous probe drilling from the tunnel face to overcome this drawback. Besides the only one-dimensional information given, probe drilling causes significant delays to excavation. A careful risk management has to address such constraints by adequate exploration and proper measures. Robust and reliable prediction methods have to be applied, which do not disrupt the tunnelling process and yield results quickly and at moderate costs. This approach uses the Tunnel Seismic Prediction (TSP) method to identify suspected fault zones identified from surface topography and geological mapping. Once the geological risk zone is identified, a probe drilling is carried out when the concerned zone is closer to the face. In addition, site geologists continually map the tunnel sidewalls to describe precisely the geological features encountered and to classify the rock mass for determination of the rock support (Dickmann, 2012).

3.1 Rock Mass Properties Obtained from Seismic Reflection Data

The ground motion caused by reflected waves is measured with seismic sensors. For the separation of the different types of elastic waves, three-component- sensors have to be used. The travel times of reflected signals are proportional to the wave velocities within the ground and to the distance to an interface. Thus, by detection of reflected elastic waves and their corresponding travel times it is possible to deduct information about the mechanical properties of the ground. In this way, important engineering parameters like Dynamic Young's modulus, Static Young's modulus, Poisson ratio, Shear modulus, Bulk modulus and density can be derived.

Figure 1 summarizes typical compressional (V_p) and shear wave (V_s) velocities found for commonly occurring sedimentary rocks. The velocity ranges mainly due to different measurement settings and rock conditions showing that velocity is a criterion for determining lithology for general application. Sedimentary rocks show a broad range of velocity, where high velocities are typical for relatively dense (low porosity) and low velocities are typical for porous rocks. With respect to the velocity and its behaviour there are two types of sedimentary rocks. Dense rocks without pores (anhydrite, salt) show a well-defined velocity which is controlled by the mineral properties and the composition. Porous rocks with a broad spectrum of velocity range show strong influence of the porosity, pore fluid, but also contact properties of solid rock components (strong cemented rocks vs. unconsolidated rocks) and the mineral composition (sandstone, dolomite, limestone, shale influence) (Schön, 2015).

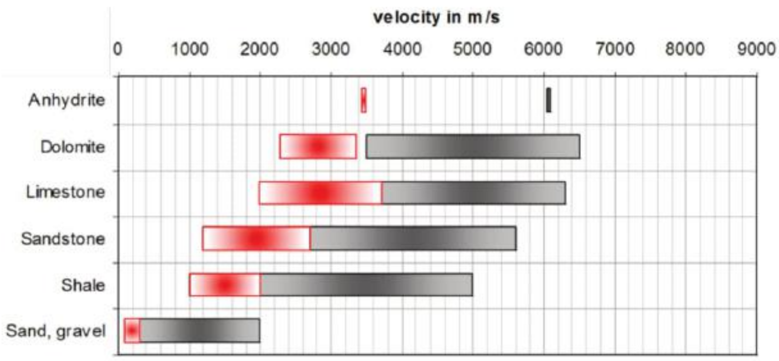

Fig. 1 - Range of the compressional (grey) and shear wave (red) velocities for commonly occurring sedimentary rocks, after Schön (2015)

The ratio of both wave velocities is controlled by the Poisson's ratio only (Fig. 2), i.e. by rock type and pore fluid. The Poisson's ratio is defined as the relative ratio of lateral strain to axial strain in a uniaxial stress state. For example, sandstone's Poisson's ratio varies from 0.16 to 0.46, whereas the higher end corresponds to highly water saturated sandstone.

The composition of an accumulated sedimentary rock varies largely because it contains the weathering product of different regions, brought to the site of deposition by water systems, wind or other mass movements. Mostly, sedimentary rocks show a bedded structure due to the way the material deposition took place.

Having the same composition, different properties of a rock and a rock mass as an assemblage of rocks usually behave quite differently. A rock sample taken for laboratory experiments might be perfectly homogeneous structured, i.e. micro-cracks, lineation and crystal orientation are equal within the inspected volume.

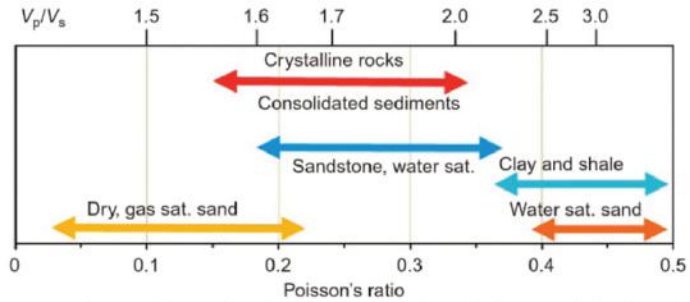

Fig. 2 - Average ratio V_p/V_s and Poisson's ratio for different lithology (Schön, 2015)

Since a typical mountain belt is usually affected by stress, large scale fractures or fractured zones are common. This, rock strength varies widely with sample size. The intact, homogeneous rock sample could have no obvious structural defects, whereas a heterogeneous and anisotropic rock mass bears all defects, which are characteristic in the field scale. In general, rock strength is usually substantial at the intact rock scale but often significantly downscaled for the rock mass.

3.2 Relation of Dynamic to Static Modulii in Massive and Fractured Rock Mass

Elastic parameters are derived from measured seismic velocities or ultrasonic measurements at frequencies ranging from >20kHz to the range of MHz, and hence called "dynamic Modulii". In contrast, the static modulii are based on 'static' or quasi static loading of rock samples and the measurement of its deformation as a function of pressure in tests in the laboratory.

Different studies show the following trends of the relationship between the dynamic and the static modulii according to Schön (2015):

(i) The static modulus is generally less than the dynamic modulus
(ii) The difference increases with increasing fracturing and porosity: most extreme differences are found in unconsolidated rocks
(iii) With increasing confining stress, the difference decreases.

Therefore, there is a strong correlation between the ratio of dynamically and statically determined modulii and pressure as well as the crack porosity for different rock types. The ratio increases as the crack porosity increases. Increasing confining pressure reduces the ratio because cracks are getting closed. Depending on rock types, the relation between the dynamic and the static modulii varies.

Sedimentary and crystalline rocks, owing to their granular structure, have intergranular cracks and micro structural boundaries. This granular micro structure is considered to be responsible for the non-linear response of the rocks. Van Heerden (1987) attributed the difference between static and dynamic modulii to the fact that rocks do not behave in a perfectly linear elastic, homogeneous and isotropic manner which is due to the presence of cracks. Cracks and non-linear response of the rocks affect the static measurements more than dynamic measurements leading to the differences in the static and dynamic modulii.

Given the nature of the rocks, it is not possible to obtain a general relation between the static and dynamic properties and hence empirical correlations have to be developed. Most of the empirical correlations available in the literature for Young's modulus have been summarized by Wang (2000). The correlations are mostly for reservoir rocks which include unconsolidated sands, sandstones and carbonates. The correlations are of the general linear form, which are as follows:

$$E_{stat} = a \cdot E_{dyn} + b, \tag{1}$$

where E_{stat} is the static Young's modulus, E_{dyn} is the dynamic Young's modulus, a and b are the coefficients whose values range from 0.41 to 1.15 for a and from -15.2 to 10.81 for b. The modulii and coefficient b are measured in GPa, whereas the coefficient a is unit less.

Van Heerden (1987) obtained a general relation between the static and dynamic Young's modulus. He found from tests at different rock types (sandstones, quartzite, norite and magnetite) at stresses of 10, 20, 30 and 40 MPa the following exponential relation between the static and dynamic modulii:

$$E_{stat} = a \cdot E_{dyn}{}^{b}, \tag{2}$$

where the values of E_{stat} and E_{dyn} are in GPa and the coefficients a and b are also stress dependent, but unit less.

In the Amberg TSP Plus software, the relation after Van Heerden (1987) is being implemented, where the overburden at face stationing is being entered and a resulting vertical stress is being considered for the relationship between E_{stat} and E_{dyn}.

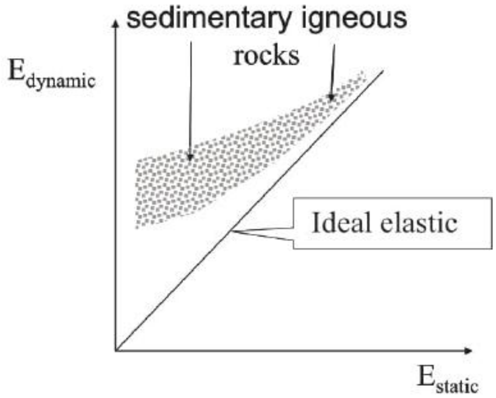

Fig. 3 - Dynamic versus static Young's modulus

4. CASE STUDY

4.1 Geology of Study Area

The project area is located in a broad tectonic zone - the `Frontal Folded Belt´ of the Himalaya - consisting of folded rocks and imbricated sequences ranging in age from Proterozoic to Pleistocene. The rock assemblage contains non-metamorphic sedimentary formations. The Siwalik Group, which dominates the broad area of the tunnels' alignment, comprises the youngest sequence of fresh water molasses sediments deposited during Mid Miocene to Lower Pleistocene time and later uplifted during the rising of the Himalaya (Fig. 4).

The proposed alignment of the tunnels is located in the Sub Himalaya, in the Miocene middle - lower Siwalik formation. The rock mass comprises of mainly sandstone, siltstone and claystone.

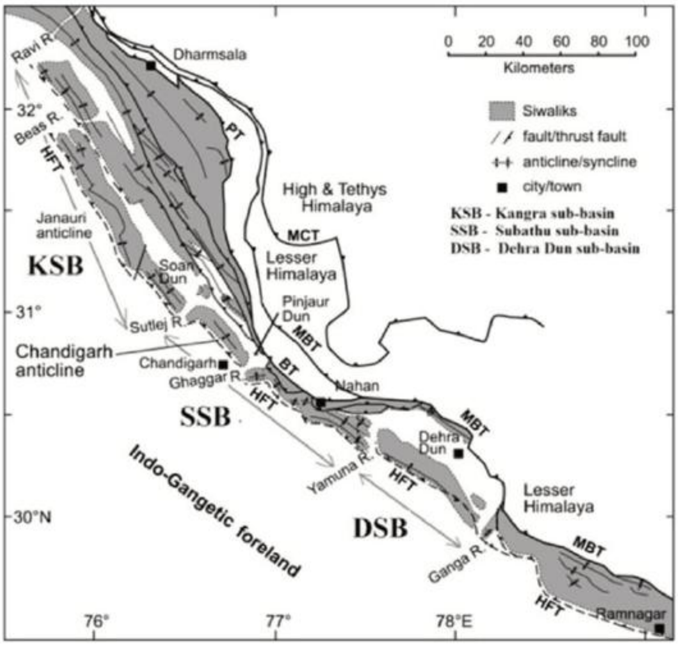

Fig. 4 - Simplified geologic map of a part of the Himalayan foreland basin (Barnes et al., 2011)

[MCT: Main Central Thrust; MBT: Main Boundary Thrust; HFT: Himalayan Frontal Thrust; BT: Bilaspur Thrust; PT: Palampur Thrust; R: river]

4.2 Objective and Approach

The objective of this case study is to derive geotechnical parameters of at least 100 m ahead of the face obtained from the seismic investigation against the encountered geology after excavation.

The measurement was carried out in the main tunnel. In parallel to the main tunnel, an escape tunnel was also under construction connecting to it by several cross-cuts. The main tunnel is 20 meter ahead of the escape tunnel as shown in Fig. 5. Here, the tunnel model exactly resembles the real geometry of the tunnel when measurement of 24 shot holes at four receivers was taken. Fixed type of delay detonators along with explosive cartridges were used for recording operation, which implied the use of Wire Break Trigger mode achieving very high accuracy of trigger time. The average overburden at the face location was about 230 meter.

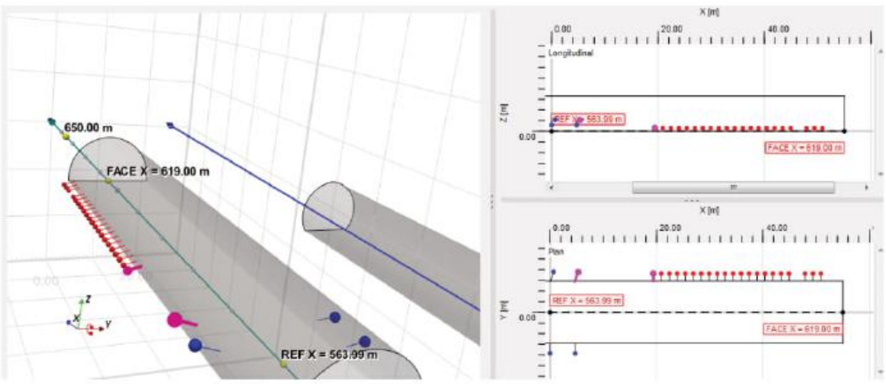

Fig. 5 - TSP layout (blue dots- 3C receivers; red dots-seismic source points)

4.3 Rock Mass Properties Derived from TSP Data

In the TSP layout, reference P- and S-wave velocities are estimated as V_p = 3,370 m/s and V_s = 1,890 m/s, respectively. These reference velocities correspond to the direct wave travelling from the shot holes to the receiver; hence, they are representative of the prevailing geology within the layout. Since P-wave velocities in sandstone vary from 2,700 m/s to 5,600 m/s (Fig. 1), it is obvious that the prevailing geology lies in the lower velocity range indicating rather poor rock. A reference E_{dyn} of 20 GPa and thereof derived E_{stat} of 7.9 GPa was estimated. Based on estimated V_p, V_s and geotechnical parameters ahead of the face, two fracture and water bearing zones were identified.

Lithology of rock mass

The geological forecast model derived and interpreted from acquired seismic data as shown in Fig. 6. Sandstone (yellow) was being assumed as regional rock mass lithology, although sandstone and siltstone formations may alternate predominantly. Both sedimentary rock types contain same minerals such as calcite, clay minerals, feldspar, micas, quartz and differ in grain size only. Siltstone comprises of smaller grains and hence smaller pores resulting in lower permeability and higher capillary pressures than in equivalent porosity sandstone. Due to different degrees of compactness, their density may differ. While Sandstone's density lies between 2.2 and 2.8 g/cm^3 depending on its fractional porosity, Siltstone's density may be little higher at values of 2.6 to 2.7 g/cm^3. Same behaviour is seen with seismic velocities, whereas water saturated Sandstone may present similar values than Siltstone. From this point of view and depending on fracturing and saturation degree, boundaries between Siltstone and Sandstone may not necessarily prove to be a significant seismic reflector.

Water bearing zone

Two potential water-bearing zones of higher ingress were predicted from the seismic TSP data. These two zones with designation "Water bearing zone" illustrated in light blue colour contain derived Poisson's ratio values of 0.33 and 0.34, respectively, wherefore higher water ingress designation has been chosen. Since a meaningful and essential calibration of TSP data could not be carried out having just one TSP campaign, assessments on levels of low, moderate and high water ingress become initially difficult.

Fracture zone

Two potential fracture zones (grey as shown in Fig. 6) have been predicted from the acquired and interpreted seismic data. Here, consideration of fracturing is based on Young's modulus values, which show local depressions in the designated areas. However, the first zone between 13+380m and 13+394m manifests more dominantly.

It is possible to view the distribution of P- or S-wave velocity, for entire area under investigation within a 3D space. In Fig. 7, 3D P-wave velocity distribution shows the already excavated tunnel tube from reference location to tunnel face as defined in tunnel model. The exact orientation and boundary of these fractured zones can be marked and verified by using the 3D viewer. The exposed anomalies have velocity less than 3,200 m/s.

In addition to other parameters, density values had been computed by empirical formula using Vp and Vs-values. Here, derived density value lies between 2.20 and 2.32 g/cm^3. These values represent average density in the given rock mass and should not be compared to grain density values that are at values of about 2.65 gm/cm^3 for sandstone.

5. COMPARISON OF TUNNEL SEISMIC PROGNOSIS WITH EXCAVATED GEOLOGY

Two potential fracture zones were being interpreted from TSP seismic data. The range of the dynamic Young modulus lies between 17 to 25 GPa as derived from interpreted TSP data. Based upon the geotechnical face mapping report, a comparison has been carried out between the parameters and observation made by geotechnical engineer/geologist and parameters derived from TSP data. According to the RMR values obtained from face mapping, rock mass within the prediction range is being classified as poor rock mass (rating 21 to 40) as it was evident also from the low P- and S-wave velocities (Table 1). The entire prediction range was revealing poor rock condition suffering lower prediction range of < 100 m. Despite this overall poor ground condition and related high wave energy attenuation, even less significant differences in the rock mass had been forecasted.

Comparing the charts of modulii and velocities with the values of rock classification (RMR), it is shown and described that the same course of curve is evident. In particular, the above mentioned fracture zones are being well predicted (compare with red zones in bottom chart of Fig.6).

In general, the validation procedure allows calibration of seismic and geotechnical parameters derived from TSP data. This procedure starts after the first one or two measurements in the same tunnel and improves the comparability of TSP derived parameters with geotechnical parameters from face mapping. Thus, an initial parameter catalogue for each individual tunnel can be compiled.

Table 1 - TSP forecast with geology as from face log derived RMR and water inflow rating

Chainage	Width of zone	TSP forecast description	Geological description by RMR and water inflow
13+371 – 13+378	~7 m	V_p increase with steady V_s → PR > 0.33 remarkable water ingress likely	Poor rock; RMR 28, 2.5-10 l/min. water inflow per TM
13+378 – 13+396	~22 m	V_p and V_s decrease to 3,060 m/s and 1,760 m/s → significant drop of dYM to 17 GPa → Fracture zone	Poor rock; RMR: 27-31 water inflow per TM from 1-2.5 l/min.to < 1 l/min
13+396 – 13+402	~6 m	V_p increase to 3,690 m/s while Vs sparsely raises → PR > 0.34 Water ingress	Poor rock; RMR: 33-37 1-2.5 l/min. water inflow per TM
13+402 – 13+418	~16 m	Rock mass little improved to moderate fractured dYM: 20-23 GPa	Poor rock; RMR: 37-33 < 1 l/min water inflow per TM
13+418 – 13+423	~5 m	V_p and V_s increase → PR > 0.29 Little water ingress	Poor rock; RMR: 35-37 1-2.5 l/min. water inflow per TM
13+435 – 13+439	~4 m	V_p and V_s decrease to 3,220 m/s and 1,880 m/s → drop of dYM to 20 GPa → Fracture zone	Poor rock; RMR: 30-28 1-10 l/min. water inflow per TM
13+439 – 13+470	~31 m	Less reliable forecast due to few reflection data. Due to persisting poor rock condition wave energy is very low.	Poor rock; RMR: 30-24 >10 l/min. water inflow per TM
13+470 –		Modulii and velocity values are increased	Poor rock; RMR: 34 < 1 l/min water inflow per TM

Notation: V_p : P-wave velocity; V_s: S-wave velocity, PR: Poisson's ratio, dYM: dynamic Young's modulus, TM: tunnel meter

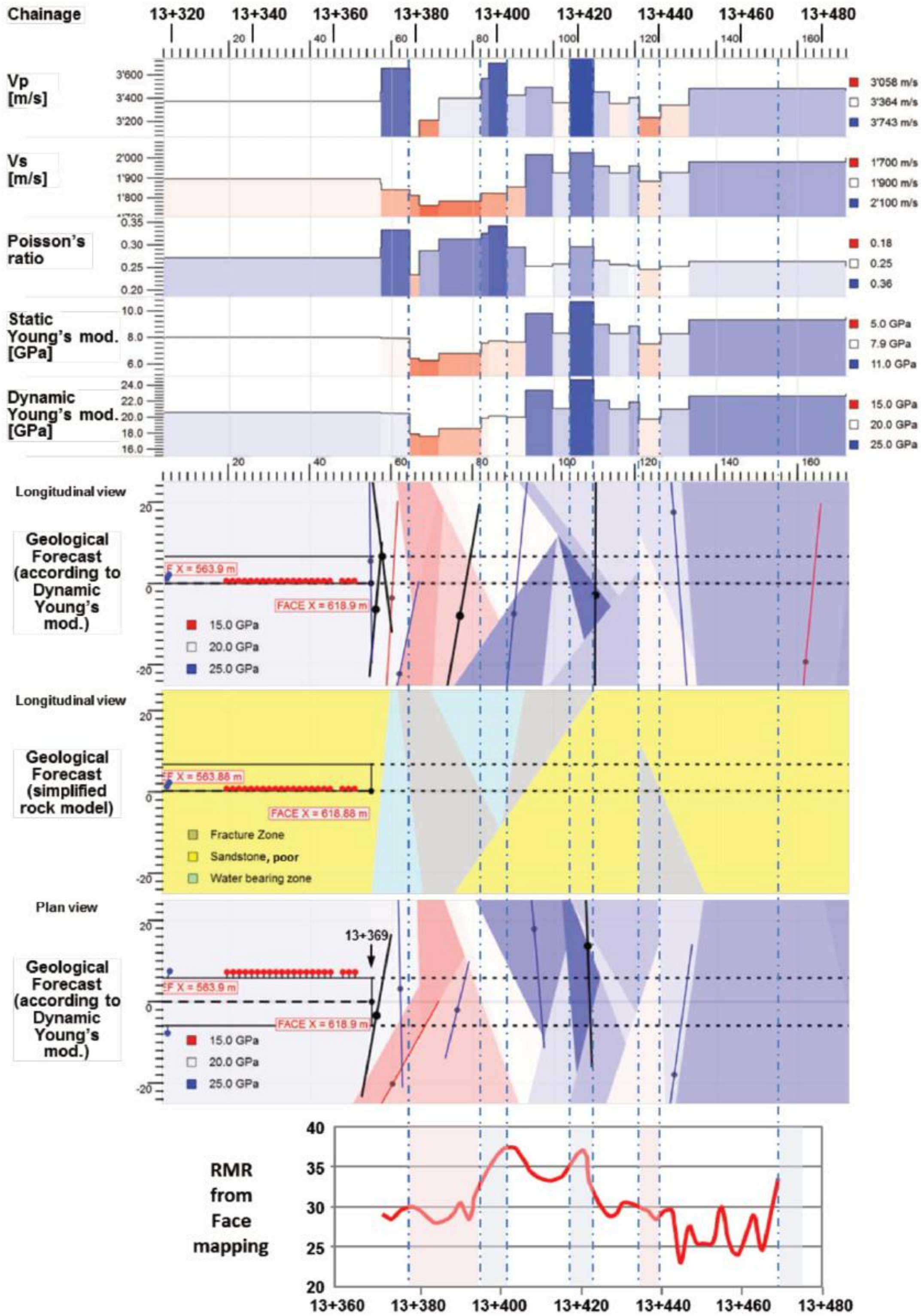

Fig. 6 - TSP charts (top) of V_p, V_s, Poisson's ratio, E_{stat}, E_{dyn} along the prediction range

Longitudinal and plan views (middle) and rock mass rating (RMR) from face mappings (bottom) are shown in Fig. 6. The blue dashed lines indicate further boundaries in the general poor rock mass, which correlate with the forecast.

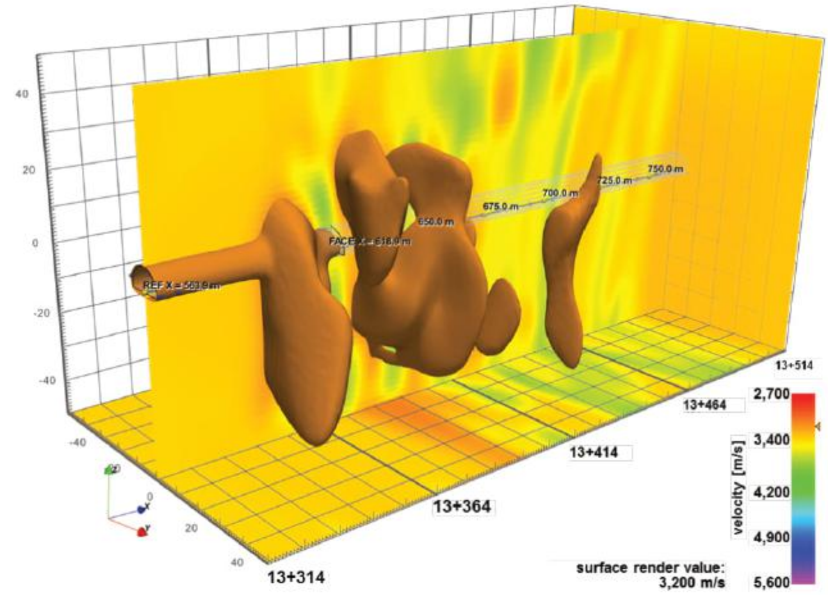

Fig. 7 - 3D-velocity distribution of the P-wave around and ahead of the tunnel face

In Fig. 7, the velocity range corresponds to P-wave travelling though sandstone (2,700-5,600 m/s) and the exposed areas represent velocity anomalies below 3,200 m/s that correspond to hazardous zones in the general poor rock.

6. CONCLUSIONS

Almost every aspect of a tunnelling project, from its conception to commissioning, is influenced by the geology of the area. Therefore, reliability of the predicted geology plays an imporatant role in the success of the project.

Geotechnical risks in the form of unforeseen geological conditions are a serious factor in cost and schedule control on all major civil engineering projects. The amounts of money involved in claims arising from these geotechnical problems are enormous and needs to take very seriously by financing agencies and engineering organisations.

As highlighted in the paper, dynamic and static Young's modulus of elasticity can be calculated and correlated with the help of advance seismic reflection survey. The TSP is found to be a useful tool giving a geological forecast that is in good agreement with observed geological conditions after excavation.

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