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Estimation of support pressure during tunnelling through squeezing grounds

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ABSTRACT

High in situ stresses and poor quality of rock mass are primarily responsible for the squeezing behaviour of rock masses. This phenomenon is prevalent especially in the Himalayan region and hence rock engineers and engineering geologists have frequently encountered problems of stability during construction in this region. High in-situ stresses, poor rock mass quality, large overburden depth and large radius or span width of a tunnel or cavern in weak rocks are the factors which are responsible for the occurrence of squeezing ground condition. The present study involves development of a dimensionally correct empirical correlation for assessment of support pressure in tunnels which are excavated in squeezing ground condition. The correlation uses the concept of 'joint factor' as a measure of rock mass quality, allowable closure, depth and radius of opening as the governing parameters. Data from 52 different tunnel sections and one set of data from a mine gallery have been considered for analysis. The predicted results have been compared with the results obtained via existing approaches, based on rock mass quality (Q) and rock mass number (N). It was observed that the proposed correlation holds better with a correlation coefficient of 0.92 and estimated values of support pressure from the approach show better accordance with the observed values of support pressure as compared to other existing correlations based on Q and N values. The proposed correlation makes use of parameters which can be easily obtained at project sites. Therefore, it can become a handy tool for site engineers to predict the support pressure in squeezing ground conditions and take appropriate measures for the stability of underground excavations.

1. Introduction

Himalayan region is full of geological surprises with regard to the underground constructional activities due to frequently changing geology (Singh and Goel, 2006). The region is highly tectonically active and squeezing of underground structures has been a major problem faced by geologists and engineers (Panthi and Nilsen, 2007). Due to this reason, the region has been a study centre of activity for many researchers. In the present study, the authors have also chosen the case studies from the Himalayan region.

Stability is the major concern in underground construction in weak rock masses due to the presence of discontinuities and high in situ stress conditions. High insitu stress or anisotropic stress condition causes rock bursting, squeezing or other stress induced stability problems (Selmer-Olsen and Broch, 1977). Stress induced stability problems in weak rock masses are characterized by squeezing. Thus, a combination of weak rock mass with high in situ stress multiplies the squeezing problem. The term '*squeezing rock*' originates from the pioneering days of tunnelling in the Alps during the excavation of railway tunnels between the years 1860 and 1910 (Kovari et al., 2000). Squeezing in tunnels is a time-dependent ground movement around the opening. This behaviour is primarily related to the progressive yielding and time-dependent deformation and degradation in the strength properties of the ground. Terzaghi (1946) described squeezing in rock as the displacement of ground under no volume change conditions. According to Barla (1995), squeezing around the tunnel opening may stop throughout the construction process or it may prolong for a considerable amount of time. According to Kovári (1998), squeezing is the phenomenon of large deformations that develop during tunnelling through weak rocks and if an attempt is made to arrest these deformations with the help of a lining, support pressure builds up and may reach values beyond the structurally manageable range. The only feasible solution in heavily squeezing ground is a flexible tunnel support system in combination with a certain amount of over-excavation in order to accommodate the deformations (Cantieni and Anagnostou, 2009).

Squeezing conditions may vary over short distances due to rock heterogeneity and variations in rock mass properties. Thus, in case of unreliable predictions of support pressure at the design stage, tunnel construction in squeezing ground becomes a herculean task claiming high cost and delay in time. However, if the support pressure can be reliably predicted using the governing parameters which can be easily assessed in the field and accordingly appropriate stabilisation measures are implemented, a good tunnelling rate can be achieved (Barla et al., 2011).

Empirical equation relating the support pressure and rock mass quality index (Q) given by Grimstad and Barton (1993) suggests that the support pressure is independent of the tunnel size, whereas the

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support pressure increases significantly with tunnel size for squeezing grounds (Singh et al., 1992; Bhasin and Grimstad, 1996; Goel et al., 1996; Bhasin et al., 2006). Empirical correlations, which have been proposed earlier for prediction of the support pressure in squeezing grounds, lack in dimensional correctness. In view of this, authors have made an attempt to develop a dimensionally correct empirical approach (correlation coefficient = 0.92) to predict the support pressure in squeezing grounds. The correlation uses data from the case histories of tunnels constructed in squeezing grounds of the Himalayan region. The parameters considered in this approach are: joint factor (as a measure of quality of rock mass), allowed radial deformation, radius and depth of tunnel.

2. Geology and tunnelling problems encountered in tunnels

2.1. Chhibro–Khodri hydroelectric project

The project was constructed on river Tons, a tributary of Yamuna river located about 45 km North of Dehradun in the state of Uttarakhand, India. Tunnel with a finished diameter of 7.5 m was constructed between Chhibro and Khodri to utilise discharge of the Chhibro powerhouse to generate 120 MW of power through a surface powerhouse at Khodri.

The Chhibro–Khodri tunnel passes through three geological Formations namely, Mandhali series (Paleozoic), Subathu–Dagshai (Lower Miocene) and Nahan series (Upper Tertiary). Mandhali series consists of boulder slates, graphitic & quartzitic slates and Bhadraj quartzite unit with 5–10 m thick crushed quartzite along Krol thrust (Figure 1). Subathu–Dagshai series is comprised of 1–3 m thick plastic black clays along the series thrust and red & purple crushed, brecciated and sheared shales and siltstones, minor grey and green crushed quartzites, 20–22 m thick black clays with thin bands of quartzites and 5–10 m thick soft and plastic black clays along the Nahan thrust (Jain et al., 1975). Nahan series is comprised of greenish grey to grey micaceous (Upper Tertiary) sandstones, purple siltstones, red, purple, grey and occasional mottled blue concretionary clays. General strike of these litho units is nearly perpendicular to the tunnel axis with the dip ranging from 20° to 60° in NNW to NNE direction (Shome et al., 1973). There are two main boundary faults running from the state of Punjab in the North to the state of Assam in the East along the foothills of the Himalayas. The faults are known locally as the Nahan and the Krol thrusts. The dips of the Nahan and the Krol thrusts vary from 27° to 30° due N10°E to N10°W and 26° due N26° W respectively. The strike is almost normal to the tunnel alignment. In situ stresses were measured using flat jack technique and ratio of horizontal to vertical in situ stresses (k) was determined to be equal to 1.

Major tunnelling problems were faced within the intra-thrust zones due to squeezing ground conditions. In order to minimise the frequent rock falls, multi-drift method was employed for construction at faces. The tunnel was excavated by drill & blast method.

Heavy steel ribs of 300 mm × 140 mm and 150 mm × 150 mm sections, with 20–25 mm thick plates welded on both flanges were erected at 0.25–0.50 m spacing to cope up with high squeezing pressures. The progress rate was tremendously slowed down to 5–6 m per month. Load cells and closure studs were installed up to 3.5 m behind the face. The observed support pressure varied from 0.65 to 1.3 MPa giving an average support pressure of 0.975 MPa in the vertical direction (Jethwa et al., 1980).

2.2. Giri–Bata hydroelectric project

This project with an installed capacity of 120 MW was constructed on Giri river, a tributary of river Yamuna. It is located near Girinagar in Sirmour district of the state of Himachal Pradesh in India. A 7.1 km long head race tunnel with a finished diameter of 3.60 m was driven through a ridge separating the valleys of Giri and Bata rivers (Dube, 1979).

The tunnel traverses through Blaini series rock formations of carboniferous age for a length of about 1500 m and through highly jointed clay stones, highly crushed phyllites and siltstones for the remaining length. The Blainis are dark grey to black quartzitic slates containing angular to round pebbles and boulders firmly embedded in a clay–silt matrix. The rock formations showed extensive jointing and shearing at places and the strike generally remained parallel to the tunnel alignment (Figure 2). Joints were spaced at 45–50 mm dipping with 60°–70°. These formations were highly crushed exhibiting an angle of internal friction between 20° and 26°. In situ stresses were measured using flat jack technique and ratio of horizontal to vertical in situ stresses (k) was determined to be equal to 2.

Most of the tunnelling problems were faced in zones of phyllites and slates. The tunnel was excavated by drill & blast method and was supported by steel ribs. Load cells and closure studs were installed up to 3 m behind the face. Blaini's slates, near the fault at chainage 1350 m also posed serious problems during construction because of high tunnel closures (>7%). Support pressure was observed in the range of 0.2–0.5 MPa. Plain cement concrete lining of 300 mm average thickness was applied as final support.

2.3. Loktak hydroelectric project

This project lies 39 km south of Imphal, the capital city of Manipur State in North-East India. It diverts 58.8 m^3/s of water from Loktak Lake to supply 16.8 m^3/s for irrigation. The remaining 42 m^3/s of water with a gross head of 312 m is used to generate 105 MW of power from three units. Finished diameter of 6.5 km long head race tunnel was 3.65 m.

Loktak tunnel traverses through lake deposits, terrace deposits and shales with thin bands of sandstones and siltstones. In the first stretch of about 830 m, the tunnel passes through lacustrine deposits. Terrace deposits were encountered in the next stretch of 420 m and the remaining part of the tunnel traverses trough splintary shales, sandy shales with variation of slaty and phyllitic types and some sandstones under

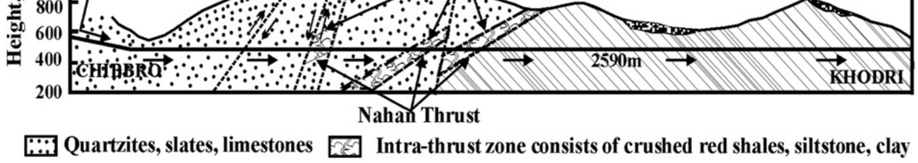

Fig. 1. Geological cross-section along Chhibro–Khodri tunnel.
After Jain et al. (1975).