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Assessment of Rock Pressure for Tunnels in the Himalayan Region — A Case History

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ASSESSMENT OF ROCK PRESSURE FOR TUNNELS IN THE HIMALAYAN REGION - A CASE HISTORY

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ABSTRACT

Since early sixties numerous tunnels had been planned and constructed for hydro-electric power generation in the Himalayan Region of India. The mountain chain is of very recent origin geologically and is believed to be still active tectonically. The geology is complex and tunneling under such circumstances had been a real challenge. It was a tough task to predict the geotechnical behaviour of the tunneling media. Beginning with the classical approach of Terzaghi (1925-46) many rock pressure estimation theories had been evolved for prediction and estimation of rock pressure for designing competent and stable tunnel supports. The author has made an attempt to project real field data, which is rare to find, during a period of over thirty years of his association with the construction of numerous tunnels driven through diverse rock formations. Various theories in vogue for rock pressure assessment yielded different results. An attempt had also been made to reason out the differences. The rock pressure assessment is still a dark area, shaded darker when dealing with weak and very weak rock formations. It is believed that earthquakes affect the surface structures most but the sub surface structures are less affected. Some earthquakes occurring in India support this notion.

INTRODUCTION

Himalayan region of India had been and still is an area for the development of Hydropower. This region holds about 80% of the hydropower potential in the country. A large number of run of the river projects had been constructed involving tunnels as water conductor system. Geologically, Himalayan region is a new mountain chain developed in last 25 million years and still the process continues. The Himalaya is therefore considered tectonically active and this may influence the stability of any engineering structure constructed with in the region. The current state of art on the subject of rock pressure estimation for designing the tunnel supports is inadequate and there is no definite approach available for realistically assessing the rock pressure. The classical analytical, modern classification and observational approaches are in vogue currently leading to some success in reasonable assessment of rock pressure for the design of tunnel supports. The author had conducted studies in a number of tunnels over thirty years that had been built in the past. An attempt has been made in this paper to project this experience. The short comings and a possible way ahead are also touched upon.

CURRENT STATE OF ART

Analytical, empirical and observational approaches in use currently are in a state of progressive improvement. Rock masses, their complex nature, effect of in situ stresses and impact of unforeseen dynamic forces of earthquake on their behaviour impose serious restriction on evaluation of a unified approach for predicting the rock mass behaviour. In the following paragraphs the approaches in currency are being discussed in brief.

ANALYTICAL APPROACH

Rock mass is assumed to have a certain brhaviour model. The most acceptable theory governing any material behaviour mathematically is the classical theory of elasticity. The commonly occurring rock mass does not follow this theory. The rocks usually exhibit a combination of visco-elasto-plastic behaviour. It is difficult to evolve a nearly adhering material model for the rocks occurring in natural conditions. The theory of elasticity is therefore applied to understand the behaviour of rock mass only in qualitative terms. The real values for rock pressure or deformations are therefore cannot be obtained for a given situation.

The theory may give good results in the case of fair to good rock masses but the situation becomes complex while dealing with poor rock formations which are very common in the Himalayan region. Terzaghi [1925] initiated the concept of assuming tunnel as cylindrical opening for stress analysis. Westregard [1940] applied the assumption for studying the stability of bore holes (small openings). Fenner [1938] considered elasto-plastic stress distribution around failing rock mass for a circular opening and proposed a theory for calculation of rock pressure. Labasse [1949], Kastner [1962], Mohr [1956], Richter [1966]. Daemen [1975] did further studies and finally helped in evolving a situation of development of broken zone around a tunnel opening. The broken zone and its behaviour govern the development of rock pressure.

With the advent of computers the computation process has got lot more simplified and made faster. Universal Distinct Element Code (UDEC) and the further developments to cover 2D/3D problems have helped greatly in developing a near natural material model for rock mass.

Rock Mass Classification Approach

Popularly known as empirical approaches had also began with Terzaghi [1946] considering discontinuities and fractures in the rocks for predicting rock pressure. Protodykanov [1963], Rabcewicz [1969], Deere et al [1969] and Muller and Sharma [1973] helped in its further development

Geological details which usually are in narrative form and not very well understood by the engineers helped in evolution of quantitative approach. A geological descriptive parameter had thus was given a numerical rating. Bieniawaski [1973] and Barton et al independently developed numerical rating based concept. Bieniawaski proposed to use the classification for support design; however, Barton evolved a formula for estimation of rock pressure. The approaches are growing with time and usage.

Observational Approach

How far the approaches described above may be helpful in estimating rock pressure under a given condition are to be proved under a given field condition. Experiments are done in the proto-type tunnels to monitor rock pressures, tunnel wall deformation and the effect of progressive tunnel driving on the surrounding rock mass. Rock pressure (loads on supports) can be assessed by installing load measuring devices (load cells) in a specially fabricated steel arch support in a tunnel. The displacement of tunnel wall can be measured across two diametrically opposite tunnel walls by installing closure bolts and measuring the progressive closure with tape extensometer. Bore–hole extensometer (multi point/single point) are installed deeper inside the surrounding rock mass of the tunnel. radially to measure the rock mass movement at different depths from the tunnel wall surface. The observed data can be compared with those obtained with the help of the approaches discussed above. The observational approach is some times followed in the proto type tunnel; however this interferes with the work progress but accepted with a view to generate the data for design of tunnel supports and to enrich the current state of art.

EXPERIMENTAL STUDIES

The author conducted experiments in some tunnels in the Himalayan region. Over a dozen tunnels were selected for experiments but detailed studies could be done in only two tunnels. , namely Giri-Bata and Chhibro-Khodri. They both lie in the Lower Himalayan region and were to pass through two major thrust zones. Dube et al [1986] discussed in details the results of the study. In the following paragraphs only relevant portions are being included.

Giri-Bata Tunnel

The tunnel lies in the tertiary rocks of lower Himalaya. The area experienced intense tectonic activities in the geological past and two nearly parallel thrust zones were detected in the area. Separation between the thrusts was believed to be about 250 metres. The rocks were phyllites/slates and clay/silt stones where the experiments were done. The proto type tunnel was 4.2 m in diameter. The studies were limited to determining the following

1. Tunnel wall and support displacements.
2. Radial displacements with in the surrounding rock with the single point borehole extensometers installed at three different depths (2.5 m, 5.0 m and 7.5 m)
3. Rock load on tunnel supports.

Regular observations of the installed instruments were taken for a period of about six months. This time is reasonable enough for the excavated tunnel to stabilize. Instruments also get unrealiable. Rock mass classification approach was used to assess the rock mass behaviour and rock pressures.

Chhibro- Khodri Tunnel

The experiments were done in a test gallery, 1.5 m radius, driven through red shale, black clays and sand stones appearing together with in the cross-section of the test gallery. The instruments were installed here were similar to those installed in the above case. Jethwa [1981] described the experiment, data analysis etc in details.

Geotechnical data for applying the rock mass classification approach was also collected while conducting the experiments.

Daemen [1975] dealt with in details the rock mass failure phenomena and the behaviour of the broken zone responsible for tunnel closure and the consequent development of rock pressure on the tunnel supports. The analysis for the observed data in this paper was done in line with the theoretical postulates of Daemen. Terzaghi, Deere et al, Bieniawaski and Barton rock mass classification approaches were used for determining the rock pressures. Table 1 gives the details,

Table 1. Comparisons of Rock Pressures in Kg/cm sq

	Terzaghi	Deere	Barton	Observed
Chhibro				
1. Red Shale	1.6-3.1	0.9-2.1	1.9-2.6	1.7
2. Black Clay	1.9-3.1	0.9-2.1	6.6-9.1	2.6
3 Red shale Highly squeezing	5.1-9.5	2.8-6.2	2.5-5.6	6.5
Giri-Bata				
1.Very blocky slates	0.7-2.3	1.2-2.3	1.4-2.4	2.0
2. Crushed Phyllites	2.3-4.4	2.3-4.4	1.1-1.9	1.7

Rocks tunneled through were weak and exhibited squeezing ground conditions. The methodology proposed by Daemen was therefore applied to assess the rock pressure and other parameters. Daemen considers the elastoplastic analysis of the broken zone around the tunnel periphery. It was assumed that the broken zone expended progressively with the advance of the tunnel excavation. The installed supports provided reaction to volumetrically expanding broken zone and ultimately stabilized on a pressure which can be considered as the final pressure. Ground reaction curve had been plotted with the help of the rock mass data, cover pressure, theoretical rock pressure and radial tunnel wall displacement. The support reaction curve had been plotted with the help of rock pressure observed and corresponding tunnel wall displacement. The details are given by Dube [1979]. The theoretical pressure, displacement and the coefficient of volumetric expansion of broken zone ,k, had also been determined with the help of field data generated through experiments..

Besides the above detailed study, rock pressures were also assessed by empirical approaches. The results are given in the table

Table 2. Comparison of theoretical
and observed rock pressures (Kg/cm sq)

Rock type	Observed	Theoretical
Phyllites of Giri-Bata Tunnel	1.7	0.5-2.0
Red shale of Chhibro- Khodri Tunnel	1.8	2.0-14.5

DISCUSSION OF RESULTS

Results of rock pressures had been obtained for two situations, namely where the surrounding rock mass has failed due to squeezing and the other where there had been no failure of rock mass due to squeezing. Such pressures are mostly due to loosening of the surrounding rock mass. In case of squeezing ground conditions, a broken zone got formed around the tunnel. This zone expanded volumetrically. The supports had been installed to arrest this expansion. The rock load therefore got manifested on the supports which had ultimately been recorded by the load cells installed in the rib. Dube, discussed in details the manifestation of squeezing rock pressure in case of Giri-bata and Chhibro-Khodri tunnels. It was observed that the broken zone radially expanded with the advance of tunnel face. Theoretically the radius of the broken zone should be between 3.5 to 4.5 times the tunnel radius according to Daemen. Dube observed that the radius of the broken zone was as high as 8 times the tunnel radius. The observed and theoretical pressures also varied. The variations confirm that the rock mass classification approach may not yield reliable results in squeezing ground conditions. The squeezing pressures are due to failure of rocks under high rock cover and poor rock mass quality of the tunneling media.

Table 3. Comparison of Estimated and Observed
Rock Pressures in Kg./cm. sq.

Tunnel	Rock	Terzaghi	Barton	Observed
Giri-Bata	Blocky Slates	0.7-2.3	1.4-1.9	2.0
	Crushed Phyllites	2.3-4.4	1.9-3.7	1.1-1.9
Tehri Dam	Phyllites Gr. I	0.0-1.6	0.8	0.1- 0.4
	Phylites Gr. II	0.0-2.8	2.1	0.5-1.1
	Phyllites Gr.III	--	1.0- 2.5	0.3-0.4
Salal	Blocky Dolomite	1.7-2.3	0.4-1.3	0.2-0.4
	Jointed Dolomites	2.3-7, 4	1.3-2.1	0.2-2.7
	Shattered Dolo	7.4	2.2-3.0	0.2-2.3
Maneri-Bhali	Quartize	0.3-0.8	0.5-1.2	0.6
	Metabasic	0.3-0.8	0.4-0.6	0.8
	Sheared Metabasics	0.9-2.9	0.8-2.1	2.0
Rihand	Granite/ Gneisses	0.0-0.7	0.1-0.8	0.1

Table 3 depicts the observed and the predicted rock pressures with the help of various rock mass classification approaches. It had been assumed that the pressures were mainly due to loosening of surrounding rock mass. The observed pressures are generally lower than to those predicted. They are there fore conservative. The observed pressures are lower due to the fact that the installation of support containing load measuring devices (load cells) takes some time for being placed in position. This time lapse is inestimable. .In the process rock load already manifested could not be recorded.

EARTHQUAKE EFFECT

The effect of earthquake had not been accounted for in any of the approaches. Bieniawaski and Barton et al accepted the insitu state of stress as a parameter in their respective classification. The earthquakes are difficult to predict. The time of occurrence, place and magnitude are beyond comprehension so far. The design of on ground and in ground engineering structure is therefore done on the basis of credible earthquake magnitude. In India the entire country is divided in five zones. The tunnels referred to in this article lie in Zone V where the credible earthquake magnitude had been assumed to be about 7.5 on the Richter scale.

It is believed that the underground structures are less vulnerable to earthquake as compared to their surface counter parts. There is no theory available to prove this so far. .The earthquakes recorded in last forty years in India support this belief. Koyna Earthquake (1967) occurred in the area where power house cavern for stage I was already constructed. The earthquake had a magnitude of 7.0 on the Richter scale and it did not in any way affect the stability of the cavern. Latur earthquake of 1993 had magnitude of 6.4. It also did not harm in any way the numerous underground structures of the existing Koyna hydroelectric power complex built by that time. The epicenter of this earthquake was about 100 km. away from the Koyna Complex.. The Uttarkashi earthqake of 1991 occurred very close to the Maneri-Bhali Stage I tunnel. This earthquake had a magnitude of 6.5 on Richter scale. The numerous underground structures for Tehri Dam project were under construction at the time of this earthquake. The site lies about 50 km from the epicenter of the earthquake. These earthquakes brought devastation to the surface structures like buildings, bridges and slopes along hill roads, but spared tunnels.

These are mere observations and some how support the notion that underground structures are less prone to .earthquake effects. This issue is therefore a topic for further research.

CONCLUSIONS

The studies referred to in the paper indicate that the various methods for estimation of rock pressures are inadequate to predict rock pressures with reasonable accuracy. The classification approaches may be good for loosening pressure conditions but they may not be good for application in squeezing ground conditions. The earthquakes are believed to be less damaging to the subsurface structures. This notion needs further research.

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