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Instrumentation and Monitoring of Underground Excavations

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"When utilizing past experience in the design of a new structure we proceed by analogy and no conclusion by analogy can be considered valid unless all the vital factors involved in the cases subject to comparison are practically identical. Experience does not tell us anything about the nature of these factors and many engineers who are proud of their experience do not even suspect the conditions required for the validity of their mental operations. Hence our practical experience can be very misleading unless it combines with a fairly accurate conception of the mechanics of the phenomena under consideration."

-Karl Terzaghi

Introduction

The above quotation from Karl Terzaghi is fairly old but valid and relevant even today. In fact, because we depend so much on our past experience without rationalizing the mechanics part of it, we are continuing to over design our structures. Terzaghi had also stated once that we all over-design because we all lack knowledge in this area and there is no way out. Even after 70 years of Terzaghi making this comment, we do not fully understand the mechanics of load development in the underground excavations, yet.

What is the way out? How do we advance state-of-the-art in this area.

We have to observe the behavior of excavations and monitor their performance before a theory can be proposed. Again, visual observations can be misleading and data obtained by careful instrumentation and monitoring only can be trusted. Once we have reliable sets of data and all the other supporting information, like the schedule of construction as actually carried out, the rate of advance, rock mass properties like in-situ stresses, modulus of deformation, strength etc., one can think of analyzing the geotechnical scenario and comparing with the observed behavior. Once such information in substantial quantity becomes available, one can think of generalizing and proposing a theory. Again quoting Terzaghi who defined the theory in the following words.

"Theory is the language by means of which lessons of experience can be clearly expressed."

Ancient Origin

We tend to think of Rock Engineering as a modern discipline, but some of the works involving Rock Engineering were carried out by Egyptians and Indians thousands of years ago. Surprisingly some of the techniques used by Egyptians for drilling, sawing and quarrying were not much different from what we use today. Similarly selecting the appropriate rock mass out of several lava flows by Indians and then carving out huge temples at Ajanta and Ellora are a testimony to their knowledge and skills. It seems that they had very good knowledge of rocks but did not understand the mechanical behavior of rock masses so well, particularly the concepts dealing with stress distribution and their long term effects.

Director AIMIL, New Delhi

Underground Excavations in Ancient Egypt

Large underground excavations have been carried out by human beings for a long time, particularly in countries where ancient civilization had flourished. Some of these have survived the test of time and can be seen even today. For example, the excavations in Egypt, Jordan and India are either called the 'wonders' or are recognized as monuments of international Heritage. The temples made by Ramses II, the great king of Egypt at Abu Simbel, though displaced by the reservoir of Aswan high dam, can still be seen in all its grandeur (Photo 1)

Photo 1: The statues and temples at Abu Symbel in Egypt constructed in 13^{th} Century before Christ

Old Quarries in Aswan

When the great pyramids were being constructed in Egypt, in addition to using the limestones available in the vicinity they thought of getting good quality granites from Aswan for use in the pyramids. The famous obelisks were also made from granite at Aswan. One of the largest Obelisks which was left incomplete at Aswan can still be seen there. It also shows the techniques used for excavation, removal from the quarry and transportation by boats to the sea and beyond. These Obelisks were made with four sides which gently tapered into a pyramid on at the top. There was one pair at Luxor as the obelisks were made in pairs at that time. In 1829, one of them was gifted to France by the then Viceroy Mr. Mehmet All. It was transported and erected at a very prominent place in Paris called Concorde Square. The way it was transported from Luxor to Paris is also depicted on the Obelisk. They did not carry the second one and in the year 1990, the Government of France decided to gift it back to Egypt. The innovative techniques used by the Egyptians at that time can still be seen and open the eyes of some of our experts of today.

Selection of the Escarpment and Lava Flows for Ajanta and Ellora Caves

The locations of the caves from the point of view of the selection of site (the escarpment, photo 2) and the particular lava flow for carving out the statues gives an indication about their knowledge of rocks. Also the technique of fracturing and cutting rocks with precision leading to such a close finish is astonishing.

Photo 2: The Escarpment housing the caves at Ajanta and Ellora

But their understanding of the 'mechanics' was weak is indicated by their design of underground caverns which were just like structures in the open. There were columns, beams, purlins similar to wooden rafters on top of which there was a slab. This can be seen in Photo 3. Also they had shown a number of elephants carrying a heavy chariot. Parts of the elephants were inside the chariot and parts were outside it. As rock cannot take any differential stresses for a long time, it was not a good idea. As a result of it, parts of the elephants have worn off (photo 4). If

Photo 3: Underground Excavation

Photo 4: Distress due to Stress Differential

one looks at the temples from this point of view, one would find several other instances where the design or the shape or both could have been altered.

Rock Material, Rock Mass and Rock Mass Classification

In Rock Engineering we distinguish between rock materials and rock masses. To be precise, we can define them as follows. Rock is a compact, natural material which is composed of one or more minerals and requires drilling, blasting, wedging or application of force in any other form to excavate. Rock substance is the solid material which does not contain obvious structural discontinuities and which can be sampled and tested in the laboratory. It is also called the 'intact rock' or the 'rock material'. Rock mass is a complex system of natural rock material comprised of blocks of intact rock and structural discontinuities that allow interaction among the blocks; too large and complex to sample and test in the laboratory.

Whereas it is possible to collect representative samples of different rock materials and test them in the laboratory, it is the sampling and testing of joints, interfaces and shear seams which poses problems. Similarly the testing of rock masses on sizes big enough to be representative, pose problems of logistics. This has led to the development of empirical relations for the quality, strength and deformability of rock masses. Unfortunately these correlations have not been tested on an actual field scale to instill confidence in the users.

Rock Mass Classification systems, particularly those developed by Bieniawski and Barton have become very popular. Whereas these systems are very good for making an initial estimate or preliminary designs, carrying them too far defeats the purpose. Since the data base or the tunnels used for developing the correlations were not instrumented, it tends to give heavier supports or more steel than what is required.

This is why instrumentation and monitoring is so important. If we have provided less steel than what is required, it will show up in the form of distress. If we have provided more than what is required, there is no way of knowing it, other than instrumentation. Since the tunnels used for creating the data base are stable, meaning thereby that the steel provided is more than what is required, the use of correlations developed on such a data base will also show more steel than required.

Squeezing Ground Conditions

Some of the projects, particularly in the Himalayan region have experienced squeezing ground conditions. The stresses in the tunneling media are high on account of overburden which get further enhanced because of the stress concentration due to excavation and the strength of the material is poor to very poor leading to its failure. The natural tendency of the tunnel, therefore, is to close or squeeze. To make the matters worse, the supports provided are generally the steel sets with pre-cast concrete laggings back-filled with lean concrete. The movement of the ground is so much that it very quickly catches up with the gaps available and start showing up in the form of twisting and buckling of steel supports. Some photographs of the buckled and twisted steel supports are shown as an example.

Two Case Studies

The cases of Giri Bata Project and Yamuna Hydel Scheme may be cited as a classical example. Both the Hydro power projects are located in the Yamuna Valley, in fact separated by the river Yamuna itself. Whereas the Giri Bata Project was being constructed on the river Giri, a tributary of Yamuna, by Himachal Pradesh State Electricity Board and was being designed by the Central Water Commission. The Yamuna Hydel Scheme on the other hand was being designed and constructed by the Irrigation Department of Uttar Pradesh. The relevant features of the two projects are given below:

a) Giri Bata Project

The first stage of Giri-Bata Project had an installed capacity of 2 units of 30MWs each. Besides a 160 m. long barrage and an intake regulator, the water conductor system of the project comprised a concrete lined tunnel of 7.4 km. length (3.66 m. diameter finished) through the ridge separating the valleys of the Giri and the Bata. In addition, it had access adits of about 0.8 km. length, bringing the total tunneling work to over 8 kms. The tunnel was passing through a geologically complex formations including three major thrusts viz. Renuka, Krol and Nahan lying in close proximity of one anoter. Rocks of different ages had come together as a result of large scale movement resulting in faulting and cutting up by numerous joints and occasional crushing. The alignment crossed two major regional thrusts viz. Krol and Nahan. As the zone in between the two thrusts was expected to be problematic, the original straight alignment of the tunnel was changed to a curved one to reduce the length of the tunnel in the intra-thrust zone. It was anticipated that the length of the tunnel in the poor material will reduce from 1000 meters to about 400 meters.

It was observed very early during construction that there were large movements and squeezing conditions. A number of experiments were carried out to contain the movement. It was seen that the use of a compressible back-fill in the space available between the rock face and the rib gave some relief to the support system, but it was only a matter of time, when the closure caught up with the empty space and thereafter the backfilled sections were equally affected.

The Closure Observations: The horizontal closure of the ribs was observed daily with respect to the day of installation. Te average closure was found in the range of 250 – 300 mms, the maximum being of the order of 500 mms. The tunnel showed appreciable closure in all reaches, irrespective of the type of support and the type of back-fill. The rate of closure was very high in the first 20 – 30 days of the installation of the ribs. Typically, the closure was 260 mms. At the end of 20 days in the case of the concrete back-filled ribs, which rose to 280 mms at the end of 75 days.

In the case of reaches with loose back-fill, the closure took place more gradually, in a more uniform manner, though the ultimate magnitudes were of the same order. In some reaches, the closures continued even after 300 days, though at a much reduced rate. Figures 1 and 2 show the typical closure observations with respect to time, for reaches with loose packing and concrete back-fill respectively.

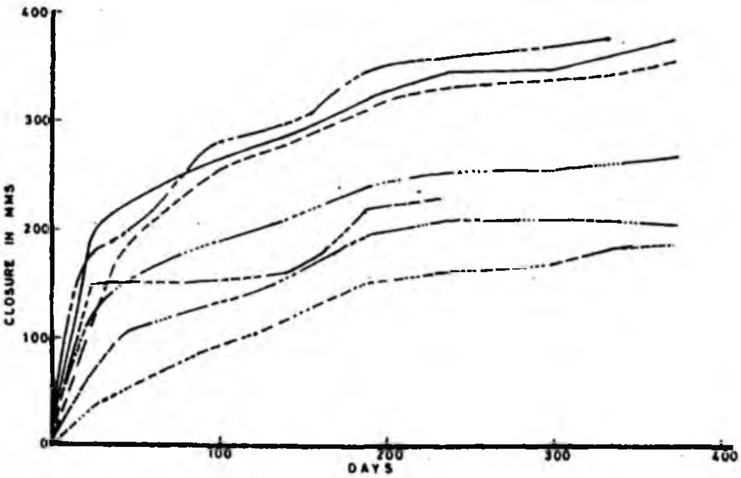

Fig.1: Closure vs. Time curve for a zone with loose packing

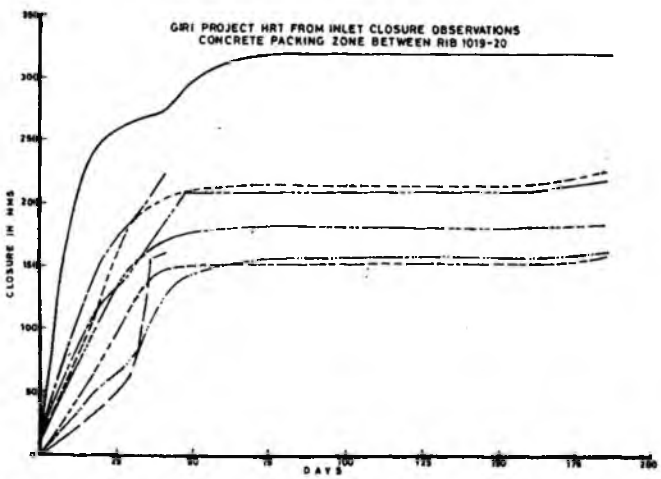

Fig. 2: Closure vs. Time curve for a reach with concrete packing

Not a Good Idea: The philosophy used by the designers at Giri Bata project was to let the deformation take place and also let the supports buckle. This led to a lot of loosening and reduction in diameter. When they started widening the tunnel to bring it to its original diameter, it started a series of destabilized zones with chimney formations. The buckled steel supports had twisted so much that some of them had to be cut with the help of welding arcs. Ultimately it was seen that it was not such a good idea to let it deform without any control.

b) Chhibro-Khodri Tunnel of Yamuna Hydel Stage-II, Part - II

On the other side of the river was located the Yamuna Hydel Scheme. It was being constructed in several stages. Stage II (Part I) comprised of a tunnel and an underground powerhouse at Chhibro. The Stage II of Part II comprised of a 5.6km long tunnel of 7.5m diameter to utilise the 64m drop between the tail race of Chhibro underground powerhouse and Khodri powerhouse for generating 12O MW power. The excavation of the tunnel was done through:

i) a shaft at Chhibro which went down to about 40m below the river bed,

ii) an approach adit at Khodri and

iii) an inclined adit at Kalawar which was initially excavated for the purpose of investigation of intra-thrust zone material characteristics.

The head race tunnel was to pass through Nahan and Mandhali formations. In between these two formations, an intra thrust zone of about 300 m width bounded by Krol and Nahan thrusts, comprising shear red shale "Sabathu" clays was to be met along the tunnel alignment. During actual excavation of tunnel the red shale were encountered again and again in the intra thrust zone, causing grave problems in the excavation and supporting of tunnel due to squeezing conditions.

The design and construction philosophy adopted by Uttar Pradesh Irrigation Department was not to allow any deformation. They placed very heavy supports in the tunnels, reinforced with extra flanges, to see that there are no movements. In the process they were able to reduce movements considerably, but ended up with a virtually steel lined tunnel.

This was causing delays and the contractor was not willing to work.

These problems delayed the completion of tunnel by nearly six years. Ultimately, it was decided to trifurcate the major tunnel into three tunnels of small diameters in the intra thrust zone.

This was no good either.

Because, this led to a very heavily steel supported tunnel which was uneconomical, also it took so long because of the contractual problems. In hind sight, we can say that both the techniques – that is not to allow any deformation or to allow too much of deformation – were faulty. We should in fact allow deformation under controlled conditions. This can be done only with the help of instrumentation and monitoring.

However these projects provided an opportunity to learn. The following conclusions could be drawn on the basis of instrumentation data and its analysis (Jethwa, 1981).

(1) The empirical approach does not provide realistic assessment of the support design parameters under squeezing rock condition. The elasto-plastic theory provided a better assessment of rock pressure under these conditions.

(2) Stiffer supports were found to attract higher load under squeezing condition. A flexible support system consisting of steel arches with loose back-fill caused significant load shedding.

(3) The tunnel closures must be considered to find out when it is safe to provide permanent supports, particularly the concrete lining.

The Phenomenon of Squeezing: It is not that the squeezing ground conditions have been experienced in India alone. These conditions have been met in European tunnels as well, particularly in the Alps and in tunnels through Mount Blanc. As a result a lot of other researchers have also applied themselves to find out the causes and work out the remedial measures.

Let us look at what is meant by squeezing ground conditions and why do they occur.

According to the International Society for Rock Mechanics (ISRM) squeezing is the time dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress (Barla, 2001). This phenomenon usually takes place in water bearing weak rocks accompanied by high tunnel depth. Most of the tunnels in the Himalayan region pass through such ground conditions and are therefore susceptible to squeezing behavior. Deformation may terminate during construction or may continue over a long period of time. The degree of squeezing often is classified to mild, moderate and high, by the conditions below,

(i) Mild squeezing: closure 1-3% of tunnel diameter;

(ii) Moderate squeezing: closure 3-5% of tunnel diameter;

(iii) High squeezing: closure > 5% of tunnel diameter.

Rate of squeezing depends on the degree of over-stress. Usually the rate is high at initial stage, say, several centimeters of tunnel closure per day for the first 1-2 weeks of excavation. Closure rate reduces with time. Squeezing, unless checked, may continue for years in exceptional cases. Squeezing may occur at shallow depths in weak and poor rock masses such as mudstone and shale, particularly if they are water charged.

Rock masses of good quality may also squeeze at great depths under very high cover or under very high tectonic stresses. Sometimes, the geological features such as syncline or the bottom of the valley may also result in high stresses.

Prof Bhawani Singh and Dr. Goel have given a simplified method in which they have separated the squeezing tunnels from the non–squeezing ones based on the Rock Mass Classification and the rock cover. It presumes that the rock mass quality is an indicator of its strength which is not necessarily true and the rock cover indicates the stress at that level. Rock cover in fact indicates the vertical stress provided the ground above that tunnel is horizontal to a reasonable extent. It can also be assumed that since the rock mass is of poor quality and it will not be able to take high differential stresses, the stresses could be assumed as hydrostatic. Within these limitations they found that the division between squeezing and non-squeezing condition is by a line H = 350 $^{1}/_{3}$, where H is in meters.

Photo 5: Heavily steel–supported tunnel section

Measurement of in-situ Stresses

In-situ stresses are required for the design of underground excavations. Unfortunately a number of techniques used in our country leave much to be desired. The most popular technique is the hydraulic fracturing. It is done in such a way by most of the organizations that we do not get any reliable data. In the absence of proper techniques, it might be worth while to estimate the stresses based on the rock cover and the topographic profile.

Photo 6: Flowing ground conditions

Analytical Methods

There has been a considerable development in the methods of analysis. In fact the present situation is that if the geometry of a problem is known correctly and the properties of the materials and interfaces involved are also known, then there is hardly any problem which cannot be solved. Some of the commercially available software have also been refined and made so user friendly that they are finding universal acceptance.

There was a time when the commercially available software were not used for research purposes and the reason was that the researchers were not sure what the black box contained. Now the software manufacturers are quite open to the question of what the software can do and what it cannot.

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