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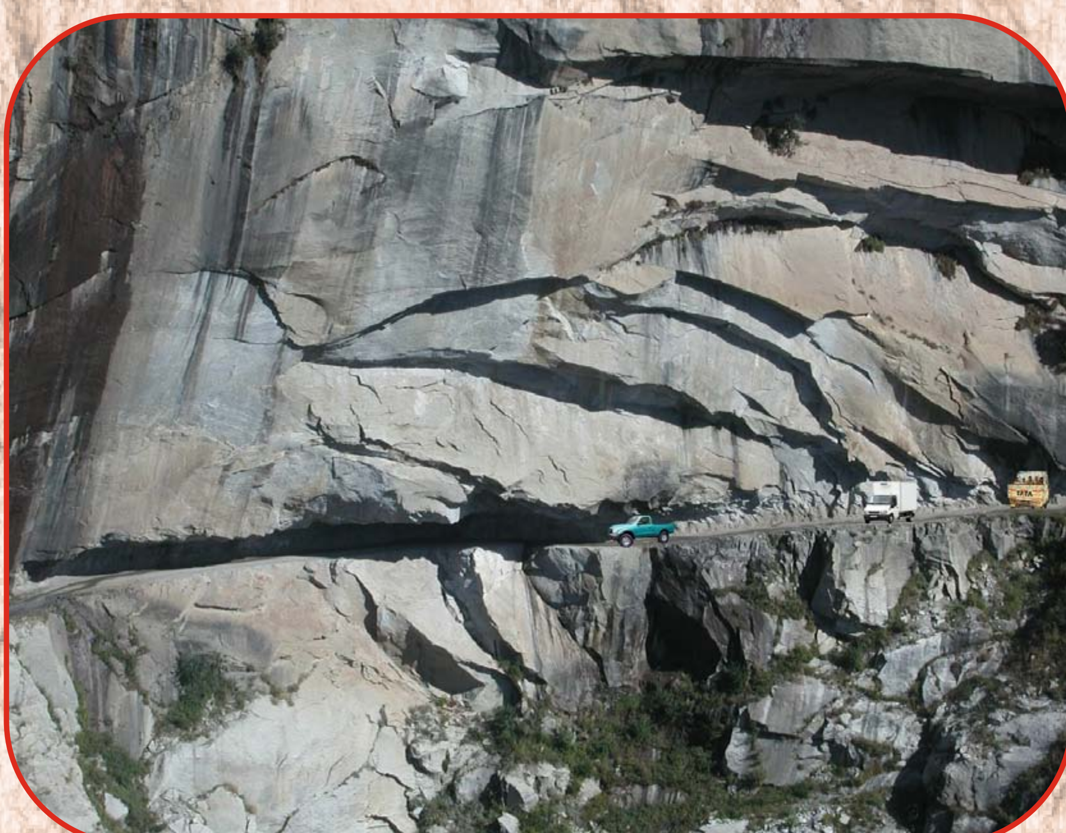

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All communications to be addressed to :

The Member Secretary

Indian National Group of ISRM
CBIP Building, Malcha Marg, Chanakyapuri, New Delhi 110 021

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FROM THE EDITOR'S DESK

First of all, I take this opportunity to wish all the members and the readers a Very Happy and Prosperous New Year.

The growth of infrastructure is the key indicator of the economic growth. Good roads, uninterrupted and quality power supply, excellent transport system, availability of sea ports, airports and railway tunnels are some of the important infrastructures, which are essential for rapid industrialization. The location of various facilities underground will be the order of day in future to ensure sustainable life. For the construction of these infrastructures, often blasting is required for excavation of the rock mass.

Blast-induced damage is an important concern to construction of underground excavations. These concerns relate mainly to the after-blast effects on stability, water inflow, safety and costs.

Underground openings like tunnels for metro lines which are at shallow depths are highly vulnerable to seismic excitations and blast induced vibrations. It was often felt that the extent of vibrations that a surface structure would experience is much more than the vibrations and subsequently the damage in underground structures. A number of case histories have proved otherwise. Although all vibrations may not induce damage to underground structures like tunnels, underground structures have indicated that damage may vary from a simple spalling of concrete to the extreme case of collapse of the entire tunnel. Such concerns have led to introduction of guidelines to regulate the extent of tolerable damage due to blasting. Editorial Board has selected a paper on "Blast Induced Response of a Tunnel in the Presence of a Two Storied Structure", for printing in this issue, which I hope will be of interest to the readers.

I thank all the authors for their contributions. I also take this opportunity to thank all the members of the Editorial Board for helping us in our endeavour and providing us with their valuable suggestions in bringing out the Journal.

I request all the readers and their colleagues/fellow professionals to contribute papers/case studies to further improve the utility of the Journal.

V.K. Kanjlia
Member Secretary
Indian National Group of ISRM

5 P SYSTEM OF EXCAVATION IN EXTREMELY WEAK AND FLOWING ROCK MASS

M.M. Madan

President & CEO (Hydro)
Jindal Power Ltd.

ABSTRACT

Tunneling through Himalayan ranges is always a challenging job especially in the weak rock formation with high ingress of water creating flowing ground conditions. Occurrence of extremely poor strata in the form of shear zones or thrust zones or highly jointed rock mass while tunneling in Himalayas makes the job of construction engineers most difficult during execution and has a greater degree of uncertainity both in respect of time and cost. Different methodologies are required to be adopted while excavating very poor rock mass (class V) and exceptionally poor (beyond class V) rock mass. Most of the time proximity of seepage water creates flowing ground. Whenever such difficult situation is encountered, it is managed by many methods ranging from primitive manual methods to modern methods using state of the art technology and latest materials. Most of the hydro projects are encountered with such tunneling problems in a small or big way.

A system was evolved while working in one of the difficult project where large lengths of extra ordinary geological conditions were encountered. The system has been termed as 5P system of tunneling through extremely weak rock formations. The 5P system is a systematic way of dealing with extremely weak rock formation with high amount of seepage water.

This paper discusses 5P System in detail and the methodology to carry out the same for tunneling through extremely weak rock mass with high ingress of water creating flowing ground. The System was successfully used in one of the projects in Himalayan geology.

INTRODUCTION

During excavation of a tunnel the rock strata encountered may include very hard rock with high standup time termed as class-I / II or a rock with good stand up time termed as Class- III or low Stand up time termed as Class- IV or very low stand up time or no stand up time, termed as Class-V or even Class-VI. The last categories when mixed with water create “Flowing ground conditions”. The strata may include; highly jointed rocks, shear zone material, thrust zone material, lake deposits, terrace deposits, bouldery strata or sandy, silty, clayey strata, overburden or glacial deposits etc. The ground starts flowing if the above strata conditions encounter seepage water or a water body or artisan condition.

Fig. 1 : Heavy Inflow of Seepage Water

Fig. 2 : Flowing Weak Ground

Fig. 3 : Extremely Weak Rock Creating Flowing Strata

THE OLD PROCESS

If the material encountered in the strata is flowing or collapsible in nature, then a plug of concrete of about 3 to 4 m thick is normally constructed at the face where flowing ground is encountered. In the concrete plug few 50 mm dia nipples are also embedded for drainage of face. In addition some nipples should be embedded on the side walls and periphery of the tunnel so that in the periphery at least 16 to 20 m long holes could be drilled in such a manner that the strata above the crown and the sides of the tunnel is well grouted to a height of 6 to 7 m. After solidifying the reach by grouting, full face excavation be carried out to a point, where the grouted strata face of about 5-6 m thickness is left for further treatment.

Grouting of the further strata is again resumed in the same way, but the nipples will have to be refixed and grouted in the required direction and grout allowed to set for three days before normal excavation work is started.

In highly waterborne strata, a number of small drainage tunnels of size about 2 sq m in section are constructed in the side wall at a reasonable distance from the main tunnel section. The percolating water will automatically flow into these tunnels, where from this water can be pumped out by sufficient pumps. These small side parallel tunnels should be excavated much in advance in heading for diverting the flow of water into these opening. In this way the strata though which the main tunnel is to be excavated will be rendered dry and excavation can be tackled by multi heading method very carefully by opening quarter heading at one time in a length of 0.5 m and installation of half side Ribs provided with vertical support. Rib supports be installed at a close spacing or zero spacing touching flange to flange depending on rock strata and thereafter immediately backfilling the space between ribs and the rock with concrete.

REVIEW OF THE OLD PROCESS

Making a concrete plug of 3-4 m itself is a time consuming process and if the strata starts flowing then it will be impossible to erect shutters and do the concreting. The ground becomes unstable and slushy and such works are carried out manually. When encountered with heterogeneous soil or gauzy material or clay type material it becomes difficult to grout and even drill holes with conventional methods.

Making of small tunnel is not an easy solution when working in constricted area and in the flowing ground conditions. In such conditions a long tunnel has to be made parallel to the main tunnel to drain out water and then normal work can be resumed. Other method is to divert the tunnel course and reroute the alignment (Tala & Allain) or reduce the dia of tunnel into smaller parallel tunnel (Giri Bata) and (Chhibro Khodri).

All these processes consumes a lot of time and money and also the non uniform zig zag alignment causes friction losses in a hydro tunnel and head is reduced to a great extent. Therefore a need was felt that there should be a methodology by which the alignment is not disturbed and the work also progresses safely and smoothly. Therefore an improved methodology was adopted.

THE IMPROVED METHODOLOGY

The improved methodology includes a system of treating the flowing ground and weak formation. The methodology is required to be fool proof and systematic in treating such a ground. It is possible to systematically treat the strata with the help of modern machines and modern materials. The improved system is explained in the following paragraph.

PLUGGING OF FACE

The first activity is the most important part in the start of the process is to provide a plug. In the condition of flowing ground it may not be possible to do the work, therefore it becomes essential to plug the entire face. The same can be achieved by placing sand filled gunny bags to provide a barrier. The bags will stop flow of muck and a stable face would be visible. With the gunny bags it is also possible to drain out water from the gaps formed in between the bags and placing drainage pipes. To provide sealing of this plug, the whole area be either shotcreted or concreted. Shotcrete is a better solution as it provides quick support. In addition steel girders can be provided for positive support. In case of weak formations sealing should be done with FR shotcrete and long face bolts. The long face bolts shall either be fibre glass bolts or self drilling bolts through which grouting can also be carried out. Further operations can be carried out after plugging the face.

Fig. 4 : Plugging of Face with Sand Bags and Reinforced with Steel Angles

PROBE DRILLING

Probe drilling be carried out to know the exact configuration of strata lying ahead of face. The probe drilling may be carried out with same machine with which drilling for pipes forepolling is carried out. It is preferred to drill a hole of 100 mm or more. A close watch by geologist should be kept on the pressure exerted during drilling and the cuttings obtained during flushing, with which prediction of the strata in the front of heading shall be made. The holes shall be drilled minimum 30m and may extend to 100m length depending upon the availability of machine and time. Three holes; one on the crown above pipe roofing, second and third on the 10 o'clock and 2 o'clock position are desirable. If high amount of water is encountered then it will act as drainage pipe or if grouting is to be carried out through the same pipe a decision can be taken accordingly at site.

PRESSURE RELIEF

Pressure relief holes shall be provided from behind the tunnel face say at a distance 5 m short of heading. These pressure relief holes will allow the water at the face to drain from the crown of tunnel away from the heading. The same machine can be used which is being used for pipe roofing for providing pressure relief. Or a drill jumbo can be used for providing pressure relief holes. These pressure relief holes may be 25 m or long depending on the pipe roofing length. The pressure relief holes shall be longer than pipe roofing.

PROTECTION OF ROOF

Protection of the roof in weak rock formations is the most important part. The protection of roof is to be done with pipe roofing or steel rebar forepoling. Steel rebar forepoling is not recommended as may not take the loand of flowing ground, or it will need larger dia of rebars which will be quite heavy and will create handling problems, therefore, it is advisable to do pipe roofing which is more load bearing and also self draining. A special machine may be deployed to do the pipe roofing operation. The machine should be capable of drilling long holes of 100 mm to 150 mm diameter upto a depth of 30-40 m at 5 degree lookout angle inclination depending on size of the tunnel. For smaller tunnels of 3m dia a hole size up to 100 mm shall be sufficient. After drilling of each hole a perforated pipe of suitable size be inserted up the drilled length. Depending on the type of strata whether collapsing or firm, the depth of hole shall be decided. For small size tunnel a depth of 10-15 m is sufficient. The pipes shall be seamless and perforated. Small pipes can be welded together to form a smooth joint. Since the same pipe shall be used for grouting operations also, preferably these should be seamless. The full crown shall be covered with these pipes. The gap between the pipes can be decided as per the site condition. For worst condition,

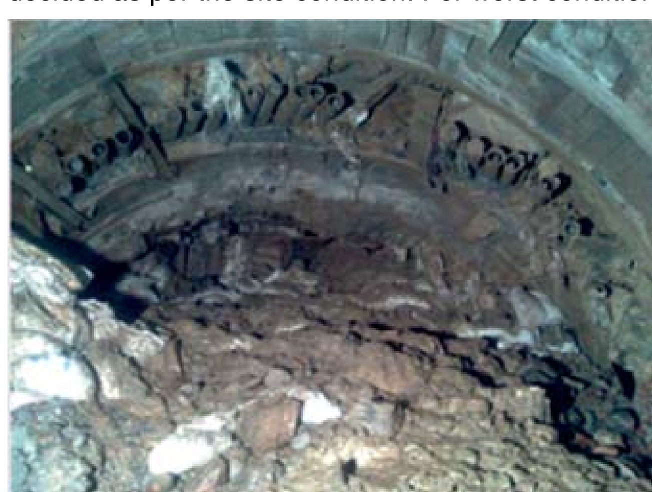


Fig. 5 : Roof Protection

no gap shall be left. For slightly better condition a gap of 150mm may be kept. If the sides are also flowing then pipes in the side wall above spring level or below may be provided. With this procedure the roof will be fully secured.

PRE GROUTING & SUPPORTING

Pre grouting of the strata beyond the pipe roofing length be carried out with OPC. The pre grouting can be taken up through the specially drilled holes below the pipe roofing in the main heading. The strata shall be grouted with non return valve type packer assembly and high pressure to allow the grout to spread. High pressure should not bulge or move the rock strata but should be sufficient to spread the grout on the entire heading at a depth of 10-20 m ahead of face, so that when excavation is carried out a better geometry of rock is found and flowing nature is controlled. If the ground is not taking OPC grout then Micro fine or ultra fine Cement shall be used with Silica Fume. In worst cases PU grout be used which may seal the water seepage instantly.

After all above actions now the face is ready for excavation upto a length 3 m short of the deepest point of pipe roofing. After excavation of the rock mass, the pipes which are acting as a roof shall be supported with steel ribs and concrete back filling be carried in a normal manner. The spacing is decided based on the type of ground conditions. Spacing between the ribs be filled with precast laggings or with fibre reinforced shotcrete.

THE 5P SYSTEM

Based on the above procedure a 5P System has been developed to excavate tunnel in the flowing ground. The system runs in five main steps which are detailed as below. The sequence must be followed as detailed.

- **1st P-Plug**

  To stop flow of ground, Plug the face with sand bags and cover the bags with thick layer of shotcrete so as to seal the face completely.

- **2nd P-Probing**

  Carry out Probe hole drilling to ascertain further rock strata by drilling three holes; one hole at the centre of the crown and two holes at 10 & 2 o'clock position. As per requirement the probe length can be decided. These holes will also act as drainage holes and shall be provided with NRV packers & pressure gauge so that in case high pressure of water is encountered the same can be controlled. If pressure is low then NRV can be removed and pipes can be used as drainage pipes and these holes can also be used for grouting of the strata.

- **3rd P-Pressure Relief**

  Provide Drainage hole for release of pressure and channelizing water flow. The place of drilling holes is to be decided as per site condition.

- **4th P-Protection of Roof**

  Provide Pipe fore-polling with seamless perforated pipes of 10-12 m or even longer length by creating an umbrella of pipes either touching skin-to-skin- or having a spacing from spring level RHS to spring level LHS depending on the rock and seepage conditions

- **5th P-Pre-grouting & Supporting**

  Carry out Pre-grouting of the strata with long holes by providing single or double packer assembly. Carry out grouting with OPC/ Micro fine cement/Ultra fine cement/Colloidal silica/Silica fumes/Chemicals/PU etc depending on ground requirement.

There after the excavation of face is carried out very carefully by opening quarter heading at one time in a length of 0.5 m and installation of half side steel Ribs and then providing vertical support. Thereafter Rib supports be installed at a close spacing or zero spacing and backfilling the space between ribs and the rock with concrete to be done immediately. As the crown is already protected, therefore it is possible to install next sets of ribs with back fill concrete upto heading for a distance short of D/2 (D is Excavated Dia of Tunnel) of pipe forepoling end. To have a support of rock minimum embedded length of pipe should be D/2. Where "D" is the excavated dia of tunnel and in case of large dia tunnels "D" is the width of pilot or width of heading, since the excavation shall be carried out in multiple haeding and benching method.

DISCUSSIONS

There cannot be rigid rules for evolving a procedure; therefore certain decisions are taken based on ground conditions observed. Some of the conditions which can be encountered are as follows;

PLUG

- During the plugging of heading, if water pressure is such that it cannot allow the plug to be put in place then Guide pipes may be placed in the plug. It will act as pressure relief for water and also it will act as guide for drilling deeper holes for pressure relief and for pre grouting of the strata in the heading.
- The plug should be leak proof, it may not be possible to do shotcreting because of leaking water, and then the face should be provided with bulkhead by doing concreting. Pipes should always be left in the bulkhead.
- If there is excessive pressure then the bulkhead

should be reinforced by providing cross girders or steel channel sections welded together with ribs.

- If it is not possible to reach the area of cavity because of fallen rock pieces & collapsed muck then the whole area which is filled with muck should be sealed with the Plug and grouted.
- It is advisable not to remove the muck and consolidation should be done in situ.

Fig. 6 : Plugged Face with Sand Bags

PROBING

- The mouth of the hole should be mounted with NRV type packer so as to take care of any high seepage water pressure.
- The water pressure should be measured by installing a pressure gauge.
- Geologist shall always be present at the face and in liaison with machine operator, the fine cutting coming out of hole shall be examined for predicting the rock type and the pressure exerted during drilling will give an idea of strength of rock strata. Geologist should be able to judge the quality of rock ahead in the tunnel face.

PRESSURE RELIEF

- Judgment has to be made whether to put pressure relief holes. Sometimes it may so happen that through the pressure relief hole all the water starts coming with a very high pressure, it becomes difficult to control the high pressure water if prior preparations are not made. (Parbati-II TBM flooding)
- A NRV should always be placed at outlet of pressure relief hole.
- If the water cannot be directed away from the work area then it is better to close the pressure relief hole by plugging with suitable material.

Fig. 7 : Shotcreted Face with Drainage and Grout Holes

PROTECTION OF ROOF

- Depending on the strata strength the spacing between the pipes be decided. In flowing ground no spacing be kept.
- NRV suitable for the dia of pipes shall be kept always ready at site.
- Dummy ribs may be required to be erected for supporting the pipes if it is observed that there is excessive pressure from the roof.
- The pipes should have a support at both ends; one on the steel rib and other in the rock at the face. The minimum support length should be half of the diameter of tunnel.
- Preferably heavy duty seamless MS pipes should be used.
- The pipes should be able to take grouting pressure of 90 kg/sqcm (9 kpa).
- The pipes should be perforated so that it can take grout or drain out water.

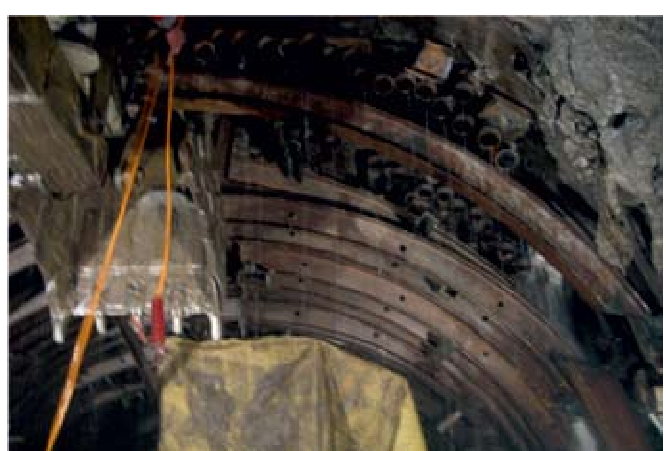


Fig. 8 : Roof Protection

PRE GROUTING & SUPPORTING

- Pre Grouting should always be attempted.
- The grouting pressure has to be controlled otherwise it may bulge the face or start movement of the ground.
- Close monitoring of the movement and leakages during grouting operation be done.
- If leakage of gout increases then the operation should be terminated and given rest for setting time.
- Water bearing jointed rocks shall always be pre grouted.
- The area under the supported roof should be excavated either by a low intensity blast or by hydraulic hammer.
- The excavated area be supported with steel rib supports and back filling with concrete or shotcreting immediately.

CONCLUSIONS

Although the tunnels are constructed in extremely weak strata but it takes much longer time to support the crown and proceed further. Sometimes it takes months of stabilization process without an inch of movement. The 5P System explained above in paper has been tried successfully at many projects in various forms but the sequence explained in the paper has been found to be most successful and regular progress can be achieved if planned in advance. All the materials and equipment should be made available in advance. Meticulous planning, quick decisions at the site and devotion of the engineers shall bring good results to expedite the excavation of tunnels with 5P System incident free.

Some More Views from Tunnels in Flowing Ground

Fig. 9 : Flooded Tunnel with Seepage Water

Fig. 10 : Heavy Flow of Water and Weak Rock

Fig. 11 : Heavy Seepage Water with Pressure

Fig. 12 : Face Supported with Channel Forepoles in Poor Ground

BIOGRAPHICAL DETAILS OF THE AUTHOR

M.M. Madan is a B Tech in Civil Engineering and MBA; He is an expert in the field of Hydro Power Development in India, construction of Underground works and Specialised in Computerized Planning and Monitoring of Large Hydroelectric Projects. To his credit he has successfully planned, constructed and commissioned many Hydro Power Projects in Himalayas.

He was Executive Director, NHPC for projects on Ravi River and tributaries, Parbati Valley projects in Himachal Pradesh, Uttrakhand projects Dhauliganga & Tanakpur and at Corporate office for Major Contracts.

Prior to Executive Director, he was General Manager & Project Head for Sikkim's Prestigious Teesta Hydro Electric Project Stage- V (510 MW), Himachal's Prestigious 800 MW Parbati Hydroelectric project Stage II and Stage III (520 MW).

He was Awarded CBI&P- I.N.Sinha Award 2002 for his contribution in Water Resources Engineering in the country and recognised as "TOP 100 ENGINEERS 2009" "International Biographical Centre" Cambridge, England

He has to his credit Publication & presentations of over 135 technical papers in professional journals, national and international seminars, symposiums and conferences.

He has published 3 Books on Tunnelling, first one was published by CBI&P, New Delhi. He was Awarded 1st Prize for Writing original Technical Book in Hindi Language by Ministry of Power, Government of India.

He is a Member of International Society of Rock Mechanics, International Tunnelling Association, International Hydropower Association, Indian Society of Rock Mechanics and Tunnelling Technology, Central Board of Irrigation & Power, Chartered Engineer and Fellow, Institution of Engineers (India) and Fellow, Institution of Valuers.

BLAST INDUCED RESPONSE OF A TUNNEL IN THE PRESENCE OF A TWO STORIED STRUCTURE

V.R. Sastry
Department of Mining Engineering, National Institute of Technology Karnataka, India

Nalini Rebello and R. Shivashankar
Department of Civil Engineering, National Institute of Technology Karnataka, India

ABSTRACT

Blast induced vibrations are common, adjacent to construction sites. Buildings adjacent to tunnel with blasting at close proximity from structures are highly vulnerable to vibrations. In this study three velocity time histories of different PPV's are applied at three different boundaries of the model. Building storey taken into consideration is a 2-storeyed building. 3 Dimensional Distinct Element code is used for the study. Response velocity spectrum generated due to blast is noted at various target points in the model. Study revealed that at higher storey the velocity and displacement increases. Displacement values indicate that models subjected to input Velocity time history at the base of model, led to greater displacement at various points of the tunnel and columns of the building compared to velocity time histories applied at other boundaries of the model.

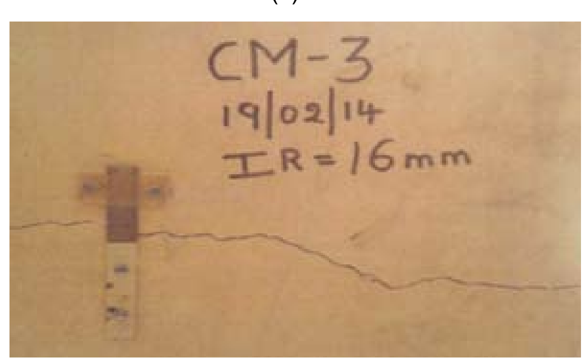
Keywords *: 3DEC, Velocity time history, Vertical displacement*

1. INTRODUCTION

Underground openings like tunnels for metro lines which are at shallow depths are highly vulnerable to seismic excitations and blast induced vibrations. It was often felt that the extent of vibrations that a surface structure would experience is much more than the vibrations and subsequently the damage in underground structures. A number of case histories have proved otherwise. Although all vibrations may not induce damage to underground structures like tunnels, underground structures have indicated that damage may vary from a simple spalling of concrete to the extreme case of collapse of the entire tunnel. A detailed explanation on the causes of tunnel failure, types and the mechanism for finding out strains is described by several researchers, Dowding and Rozen[1], Hashash et al.[4], John et al.[5] and Wang et al.[11]. Most researchers have concentrated their studies on damage assessment in tunnels subjected to seismic excitations due to earthquake loads. However, a very few researchers, (Liu[6], Lu et al.[7], Tian[10] and Wei et al.[12]) have carried out detailed numerical investigations of tunnel subjected to blast loads. Their main focus of study was finding out the PPV's at various locations of tunnel, by converting the pressure generated on blasting, to a Triangular velocity history (Stress wave) on the inner lining of the tunnel.

Blast conducted at a depth from surface, induces damages to overlying structures in the form of cracks (Fig. 1). The water tank was at a distance of 20 m from a site where an underground controlled blasting took place.

(a)



(b)

Fig. 1(a), (b) Position of crack in a Water tank close to an underground blast site

Dynamic impact loads are different from earthquake loads as they last only for a few seconds and are subjected to very high frequencies unlike earthquake loads which last for several seconds and have low frequencies. Also if we notice the characteristics of a blast wave generated due the explosion of known quantity of explosives, it is triangular in shape as illustrated in Fig. 2.

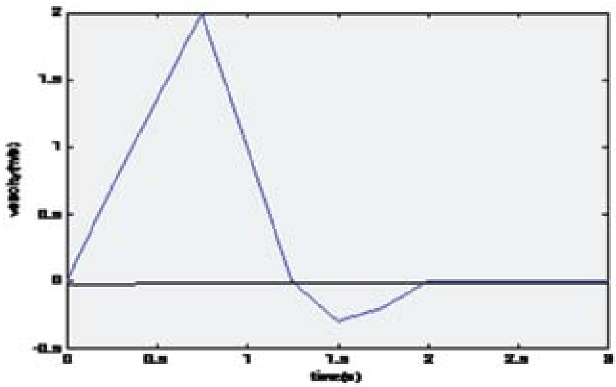


Fig. 2 : Time history generated due to a typical blast

Blast PPV's are characterized by dominant Vertical Velocities compared to horizontal velocities and therefore these vertical vibrations will induce damage to buildings which are different than that induced by Earthquake loads. Displacement generated due to Earthquake loads is characterized by inter-storey drift, between floor to floor, whereas displacement generated due blast waves produce a degradation of the different components of structure[3],[10].

In order to study in detail, the displacement generated due to vertical component of blast wave, three actual velocity histories generated due to explosion was applied at different locations of the model, and PPV's, displacement and stress concentration during the event were noted at different target points in the model.

2. PROBLEM DEFINITION

A parametric study was carried out incorporating various input Velocity histories and application of velocity histories at various geometrical boundaries, in single layer of strata, to study the following:

(a) Effect of varying input PPV's (Frequencies) on the stability of building and tunnel

(b) Effect of applying dynamic vertical wave at different locations of the model

2.1 Details Assigned to Strata

Studies were carried out taking a case study of a tunnel in South India. The tunnel crown is at a depth of -8 m from the surface. Diameter of the tunnel taken into consideration was 6.1 m (Fig. 3). Concrete segmental lining of 0.25 m thickness was provided immediately after excavation. A single layer of strata was taken into consideration. The material taken into consideration was soil of uniform density 2200 kg/m^3 and cohesion of 10 kN/m^2. Ko value of 0.5 was taken throughout the analysis. The properties assigned to the strata are in Table 1. A linear-elastic constitutive model is assigned for the tunnel liner with a Modulus of Elasticity of 3.1e^{10}Pa and Poisson's ratio of 0.1.

Table 1 : Properties Assigned to strata

Density (kg/m^3)	Bulk modulus (Pa)	Rigidity modulus (Pa)	Angle of internal friction
2200	3.3e8	1.1e8	32°

2.2 Details of the Building

A framed 2-storey building without brick-infill walls was considered for the analysis (Fig. 3). Columns are of size 0.35 m x 0.45 m with an axial stiffness of 128 MN. Slab is assigned a thickness of 0.15 m. Beams have cross-sectional dimension of 0.3 x 0.35 m with axial stiffness of 85.8 MN and bending stiffness 0.876 MN-m^2. Even though the above mentioned dimensions are characteristics of structures with more number of floors, slightly oversized beams and columns are provided to facilitate ease in modelling. The footings are of dimensions 2 m x 2 m with a thickness of 0.5 m. A distance of 4 m is provided from the centre line of one footing to the other, both in the transverse as well as longitudinal direction.

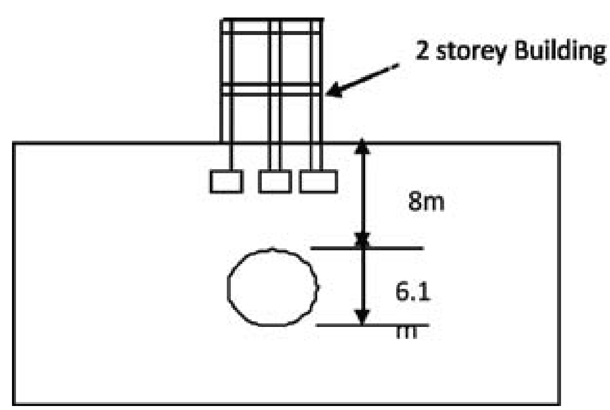
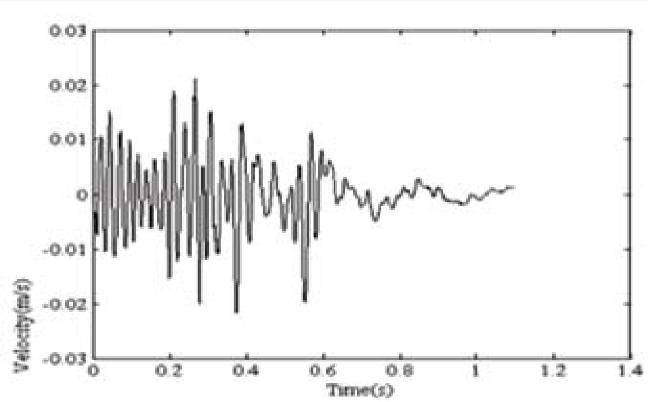


Fig. 3 : Elevation of the tunnel and building

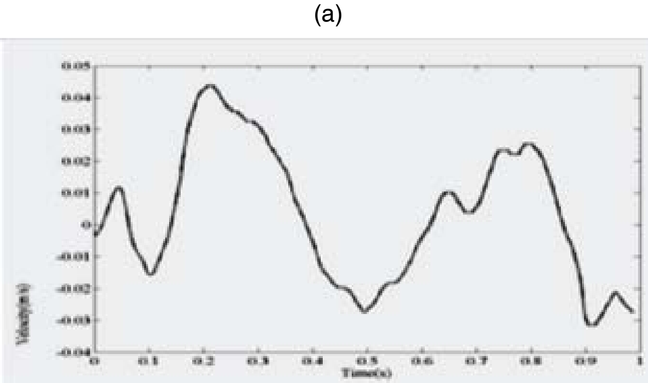
2.3 Details of Dynamic Modelling

To reduce the effect of artificial boundaries, a distance of 4D was provided (where 'D' is the diameter of tunnel) at the sides in the transverse direction and a distance of 18 m was provided from tunnel bottom to the bottom of the model. In order to prevent the waves from reflecting back into the model viscous boundaries developed by[8] were applied to the side and bottom boundaries of the model. The entire domain was divided into deformable blocks with each block further discretized into tetrahedrons. The tunnel is excavated and brought to a state of static equilibrium prior to application of dynamic loads. Later on the presence of the building with dynamic load applied at various locations of the model was analysed

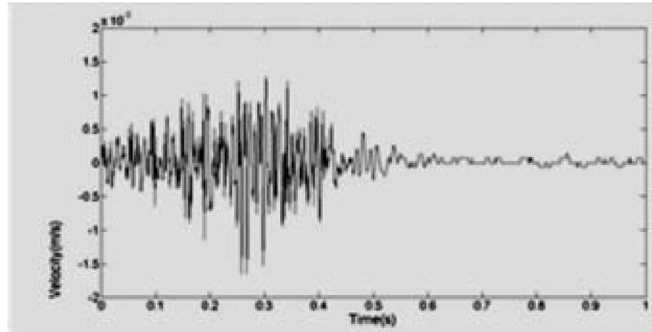
In 3DEC, the input wave may be a stress wave/pressure wave or a velocity wave. Most researchers have applied a pressure wave as an input to the inner side of the tunnel[6],[12]. However[2] has proved in his research, using discrete element method, that in layered/jointed strata a velocity time input is more effective than a pressure time history as the former type effectively transmits input waves without any reflection back into the media and therefore in the present study a velocity wave was used as an input in the model. Velocity history generated due to a blast was input at three different locations of the model. In the first case, the velocity history was applied at the bottom of the model, assuming an explosion to have taken place in a deep underground mine. In the second case, Velocity history was applied at the crown of the tunnel, assuming an in tunnel explosion and in third case, blast wave was applied at the surface(Fig. 4).Total duration of the blast is 1s and time step used for the analysis was Δt = 0.096 ms. Three different blast waves of different PPV's were used for the analysis. Thus in the first case a blast wave of PPV 21.5 mm/s and frequency of 45 Hz (Fig. 4(a)), second case a blast wave of PPV 45 mm/s and frequency of 2 Hz (Fig. 4(b)) and third case a blast wave of PPV 1.8 mm/s and Frequency 85Hz was taken up for analysis ((Fig. 4(c)).



(a)



(b)



(c)

Fig. 4(a), (b), (c) : Velocity time history applied to the model

3. ANALYSIS OF RESULTS

Results of the analysis are presented in two stages

1. Analysis of Velocity and displacement in 2-storey structure.
2. Analysis of Stresses surrounding tunnels and the building.

Prior to the application of actual dynamic loads the natural frequency of the building with the tunnel embedded at a depth of 8m was analysed. Thus the natural frequency of the two storied structure was 2.2Hz.

3.1 Analysis of Velocity and Displacement in the Structure

Vertical velocity histories generated at different floor levels indicate that they are much higher than the input wave motion when the wave was applied at the bottom of the model, followed by input motions applied at the surface and crown.

For an input wave of 45 Hz (21.5 mm/s), results of the analysis reveal that Peak velocities were dominant at the top beam with a value of 28.7mm/s, 21.07mm/s and 12.95mm/s when the wave was applied at the bottom of the model, crown of tunnel and surface respectively (Fig. 5). Vertical velocities were higher at the beam of 2nd floor compared to the beam at 1st floor level.

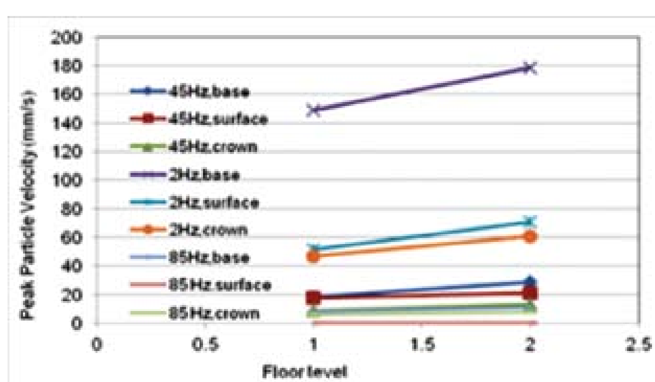
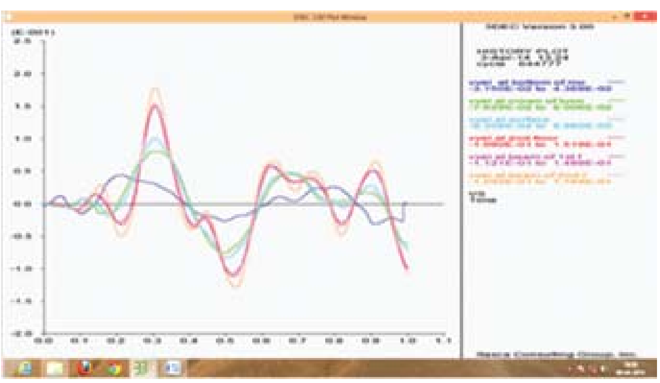


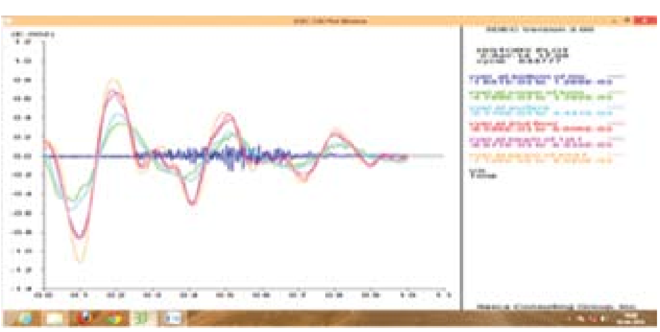
Fig. 5 : Peak particle velocities at different floor levels

Maximum magnification of Velocity amplitudes occurs in structures where in the input wave has a frequency which is equivalent to the natural frequency of the building. As can been seen in Fig. 6, PPV's at the beam level of the second floor, due to input wave of frequency 45 Hz, 2Hz and 85 Hz were 28.7 mm/s, 178.4 mm/s and 11.26 mm/s respectively indication amplification of velocity at 2Hz input frequency. The above results are in conformation with results on experimental investigations conducted by Singh et al.[9].

(a)



(b)



(c)

Fig. 6(a),(b),(c) : Velocity histories generated due to application of waves of different frequencies

Since blast waves do not produce horizontal drifts compared to vibrations produced by seismic excitations, only vertical displacement at various floors of the building were monitored.

Vertical displacement at the 2nd storey and 1st floor level due to application of dynamic input of 45 Hz, 2 Hz and 85 Hz were 0.878 mm and 0.739 mm, 104.5 mm and 100.9 mm, 0.65 mm and 0.6 mm respectively indicating that there is a linear relationship between increase in PPV's and increase in displacement.

2. Analysis of Stress Surrounding Building and Tunnels

To understand the stresses at various locations, points B1, B2, B3, T1 and T2, was taken into consideration (Fig. 7). Stresses at location T1, T2, indicates that the concentration of stress at the tunnel sides was larger than the crown of tunnel although the magnitude of stress is the same for varying locations of velocity wave input (Fig. 7, Table 3).

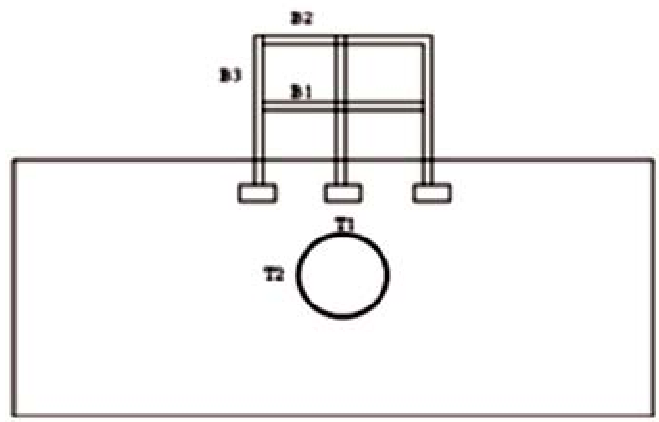


Fig. 7 : Model indicating various observation points

The concentration of stress at the side of the tunnel was 1.17e6 N/m^2, which increased to 1.3e^6 N/m2, 1.43e^{6}N/m^2 and 1.23e^6 N/m^2 respectively on the application of the 45 Hz, 2 Hz and 85 Hz wave. Peak Stress concentration at the beams of the 2nd floor was more than the concentration of stress at the 1st floor indicating that the top floors are more prone to damage than bottom floors.

Table 3 : Vertical Stress at different locations

Observation points in the model	Stress (σyy), N/m^2		
	At bottom	At surface	At tunnel top
T1	8.78E+04	8.20E+04	8.97E+05
T2	1.43E+06	1.27E+06	1.25E+06
B1	5.26E+03	4.55E+03	4.69E+03
B2	6.50E+04	4.12E+04	3.66E+04
B3	6.38E+03	9.36E+03	6.43E+03

CONCLUSIONS

1. Application of velocity history at the bottom of the model led to maximum response in the tunnel and structure compared to velocity histories applied at other points in the model.
2. Results of the analysis indicated that for a given building Peak Velocity response occur in the beams of the top floor.
3. Greater displacement generated at the building top resulted in greater stress release in beams of top floors.
4. Irrespective of the location of application of input wave, Maximum concentration of stress occurred at the tunnel sides.
5. From the above two points we can conclude that Blast related damages are more vulnerable at the beams of the top floors and the sides of the tunnel.

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ASSESSMENT OF SLOPE STABILITY WITH STRUCTURAL MAPPING IN RG OC-II, SCCL

D.N. Sharma, R. Vinay Kumar and G. Rajendar
Exploration Division, The Singareni Collieries Co Ltd., India

ABSTRACT

A detailed geological mapping, recording the data on Joints and faults to assess the stability of slopes of both Sedimentary and Metamorphic benches in one of the opencast Coal mines ie RG OC-II of the Singareni Collieries Company Ltd (SCCL) located in Godavari Khani is planned as a part of deepening the workings to a depth of 400 m and to raise the height of OB benches from the present 90 m to 120 m. In this connection, a detailed mapping is planned and generated all desired data. Mine authorities reported that 60cm of sliding is observed in Highwall towards SE-NW and is along the major boundary fault (F1). The paper deals with the Structural Mapping to assess the stability of Over Burden benches in the Opencast Coal mine. An attempt is made to compare the data on joints generated by different Institutions/agencies in different locations within the same block.

1. GEOLOGY AND STRUCTURE OF RG OC-II BLOCK

1.1 Geology

The field site is located within the Ramagundam Coalbelt along the western margin of the Godavari Basin. Up to seven regionally correlatable coal seams contained within the Lower Permian-Upper Barakar Formation are the target of coal exploration and mining. Near major boundary fault, located on western side of the block, coal bearing Barakar Fm directly come in contact with metamorphic basement rocks of Archaean group (Photo-1), thus missing a natural sequence of Pakhal Fm and Sullavai Fm. Four minable seams named from top to bottom I,II,III and IV and three thin seams, IA,IIIB and IIIA are regionally consistent over many kilometers. Within the block, to a large extent, III&IV seams are merged and formed a combined seam.

1.2 Structure

Trend of coal seams/ coal measures varies from NW-SE to NNE-SSW with varying gradient of 1 in 4.40 to 7.30. On the southern side, major fault F1 forms the limit of the block. Along this fault, older formations viz. Pakhal Fm, Sullavai Fm and Talchir Fm, were faulted and the Basement rocks of Archaen group came in contact directly with basal Barakar Fm. It is presumed that throw of the fault could be +300 m?, as natural sequence of Fms. are missing along with part of Archaen Fm (?). Apart the major fault, the block is intersected by several faults with varying throw amount.

2. PREVIOUS WORK

2.1 Sirovision Technique by CSIRO, Australia on Highwall Stability Near Punch Entries.

CSIRO, Australia taken up detailed investigations to assess the highwall stability over punch entries during Dec 2008. Using the technique of digital photographs as a part of structural mapping, with the help of SiroVision, turns the images in to high-precision 3D images of the rock mass surface. These 3D images were used to map and analyse the distribution of joints in the surface of the rock mass. This allows large areas to be mapped quickly and safely. The extract of the report is reproduced hereunder to have a comparative study on Joint pattern.

Sirojoint® has two modules - a measurement/analysis module and a data analysis module. The measurement/ analysis module is the entry screen and is the component in which 3D images are used to map structure. It also provides plotting and analysis facilities. Once the discontinuities in a 3D image have been mapped, they can be quickly analysed and grouped into sets based on orientation (automatically or manually) or domain (user defined). In addition to the orientation of discontinuities, SiroJoint® calculates the discontinuity length (maximum chord length), area and spacing. An estimate of the persistence of a discontinuity is obtained using the maximum chord length of the plane.

Preliminary analyses of the joint distribution were done on image 11a (Fig. 1). This image was chosen as it covered a larger area when compared to other images, and the exposure is good with limited rubble. The joint statistics are plotted in Figure 1.

2.2 Hade of Boundary Fault (F1)

Mapped the major fault F1 exposed in the quarry of RG OC-II during Sept. 2011, in view of the sliding of bench reported by mine authorities. All along the exposure of fault plane, number of readings of the fault angle were taken. It ranges from 52^o to 55^o and It indicates that the hade of the fault varies from 35^o to 38^o. In general, based

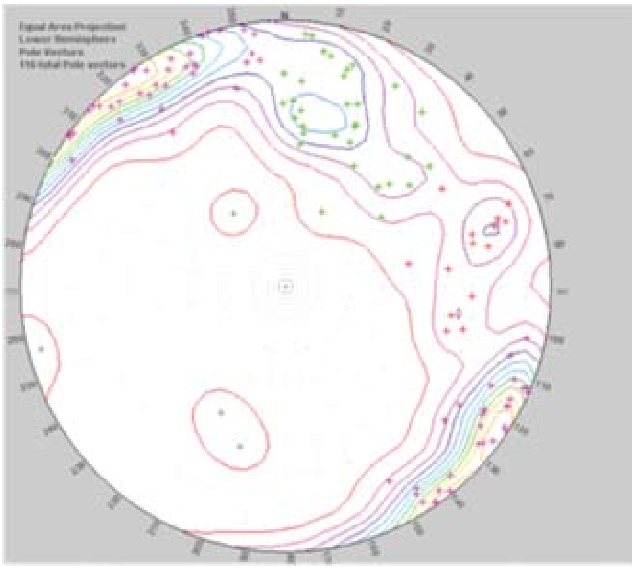

Fig. 1 : Contoured stereo plot showing joint distribution from image 11a. Poles to joints are plotted. There are three distinct clusters (Pink +, JS1; Green +, JS2; Red +, JS3). Plotted in SiroJoint®, equal area projection, lower hemisphere, modified Kamb contouring method.

on the experience, while drawing the floor contours of coal seams, the fault angle 60° (=30° hade) is taken into consideration.

In light of the above observations, it was suggested to consider the hade of the boundary fault (F1) as 35° to 38°. Further, it was suggested to look into re-design the benches to have a overall slope of highwall on the safeside.

3. PRESENT WORK

3.1 Mapping of Joint Pattern in Punch Entries in I Seam

A detailed underground Geotechnical mapping was carried out in the punch entries of Adriyala mine through RG OC-II to pick up Joints. The aim of this work is to ascertain and validate stress orientation with that of similar technique used in the longwall panels of GDK-10A Incl in predicting the Stress orientation in the dipside property i.e., Adriyala longwall block. The mapping interpretation thus made was validated with that of Hydrofracturing data generated subsequently in Longwall panel-1 of Ardiyala. It indicates that it is in a good agreement. Keeping this in view, mapping is presently taken up in punch entries to trace the Joints along the sandstone roof of working section of I Seam to further validate with the earlier established data. As a part of the work, FLAC-3D is also taken up to analyse the orientation of level and dip gallery and their stability. It is established that the roof conditions are most stable. Part of this data relevant to present studies of RG OC-II, is considered to make a comparison of joint pattern and is as follows :

A total of 54 joints have been picked up in the mapping of the punch entries. For all these joints, Rose diagram (Fig. 2) and stereographic projection of the poles (Fig. 3) are drawn. It indicates that 3 joint sets viz.J1, J2 and J3 are delineated. Among these three sets of Joints, J1 trends in N20E, J2 trends in N4E and J3 joint is in the direction of N66W. The dip direction and dip amounts have been given below :

J1 (Dip direction / Dip amount): 110.8/75.6

J2 (Dip direction / Dip amount): 273/82

J3 (Dip direction / Dip amount): 27/55

From the above observations, it is found that J1 and J2 are having more or less the same strike around N20°E but their dip directions are away from each other. Moreover, majority of the joints are having their readings matching with J1. The above findings are inline with the results of hydro-fracturing tests, wherein, Major horizontal stress direction: N24°E and Minor horizontal stress direction: N660W were reported.

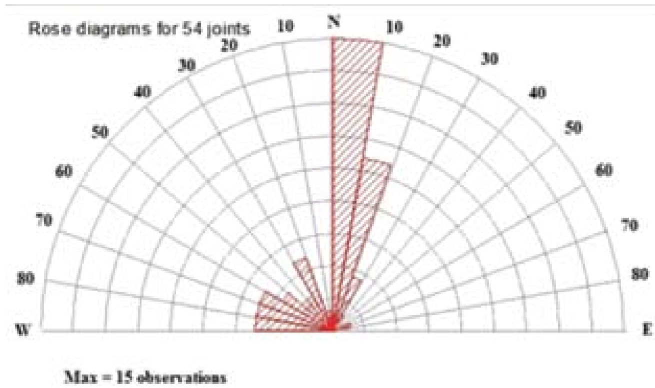

Fig. 2 : Rose diagram of Joints-Punch Entries. (Total 54 readings)

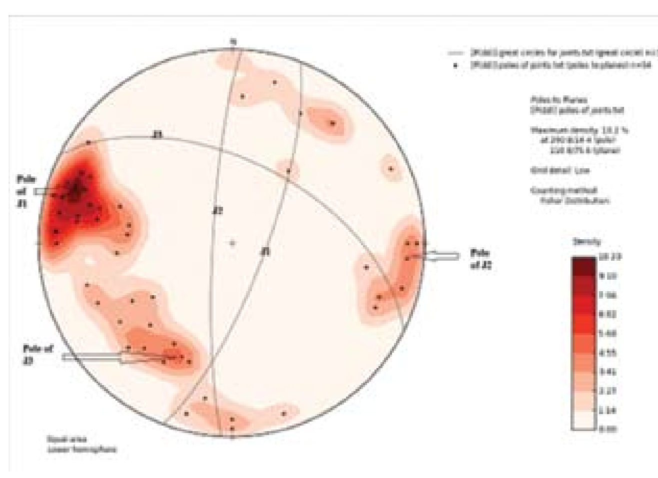

Fig. 3 : Stereographic projection of joint poles-Punch Entries

3.2 Mapping in the Metamorphic and Sedimentary Benches in the Study Area

A detailed and systematic mapping is planned to pick up trends of Joint system of both Metamorphic and Sedimentary benches along the major boundary fault (F1) to find the reasons that are leading to the wide cracks and thereby resulting into failure of benches, In the study area, mapping of major fault couldn't be taken up due to various reasons viz. exact fault plane couldn't be located on the ground since it is disturbed during de-coal, inaccessible etc. The fault was mapped and the details are discussed. Exact location of major fault was surveyed and was taken into consideration.

3.2.1 *Joint Pattern*

Total 45 joints are traced in sandstone benches (Photo 1) and 55 joint readings are taken in Metamorphic benches (Photo 2) and all put together, a total of 100 readings are recorded. Locations of these Joints are surveyed with the help of total station. At each location, the details of Joints are taken viz. trend, dip direction, dip amount, persistence, spacing, Joint filling, Joint aperture and Joint surface. Rose diagrams are drawn for respective benches of Metamorphic and Sedimentary and depicted in Fig. 4. From the Rose diagrams, it is inferred that in Metamorphic benches, mean trend of most prominent Joint set J1 is in N35°E, whereas in sandstone benches, mean trend of most prominent Joint set J1 is in N75°W. Correspondingly least prominent joint set is J2 trends in N5°W in Metamorphic benches and J3 in N35°W of sedimentary benches. It is observed that in Metamorphic rocks, only two most prominent major sets of Joints developed and the third set is not significant. Further, the Metamorphic rocks are more prone to the major fault F1 and weathering.

In the Sedimentary benches, though three sets of Joints are observed, it is interesting to note that sympathetic to Ji joints, several joints developed as evidenced in the Rose diagram. (Fig. 4).

An attempt is made to compare the data on joints generated by different Institutions/agencies in different locations within the same block. However, on comparison of Joint set data of Sedimentary benches with that of Joint set data reported by SIROVISION of CSIRO and Punch Entries studies by SCCL are varying. Further, it is observed while correlating the trends of Joints that they are controlled by Structural elements viz. Roll-over structure, faults etc and are localized and independent.

Further, Stereographic projection of joint poles on lower hemisphere are drawn using Openstereo software, the poles of the joint planes and the contours for the poles have been plotted This contour diagram is used to identify the clustering of the joint poles which is further used to identify the different sets of joints and their mean orientations. The data thus processed is depicted in Figs. 5 to 7.

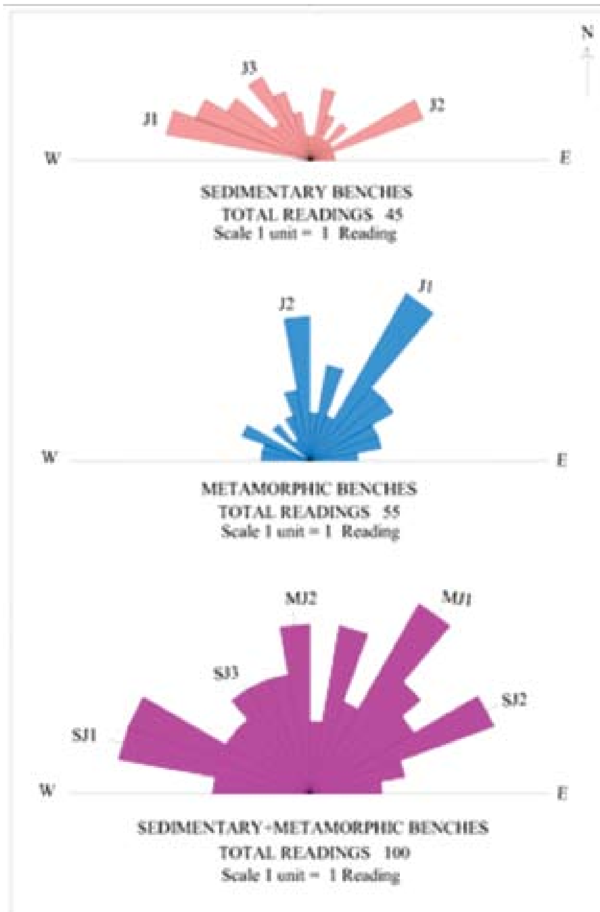

Fig. 4 : Rose diagrams of Joint pattern of Metamorphic and Sedimentary benches (MJ1= Metamorphic J1, MJ2 = Metamorphic J2, SJ1=Sedimentary J1, SJ2 = Sedimentary J2, SJ3= Sedimentary J3)

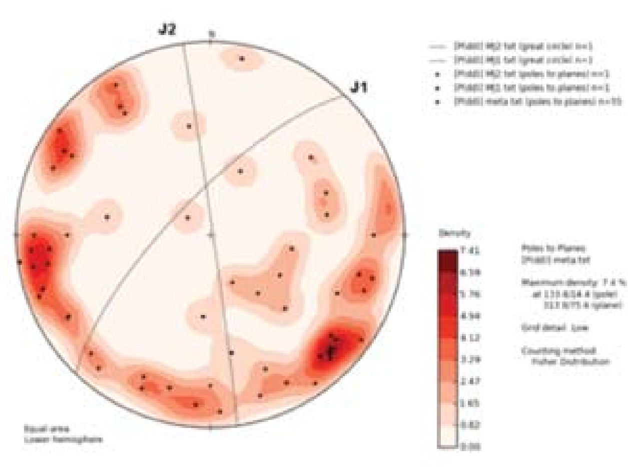

Fig. 5 : Stereographic projection of joint poles of Metamorphic benches

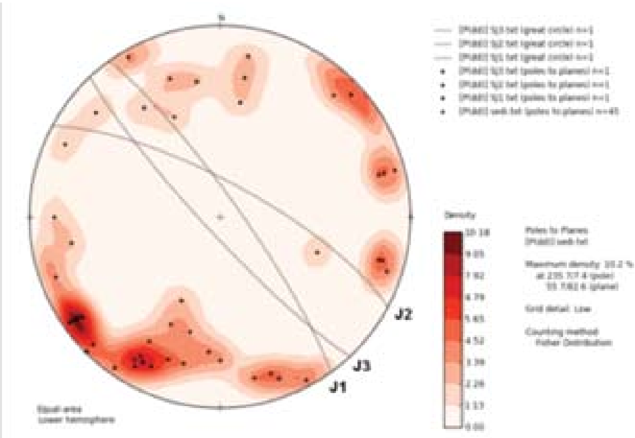

Fig. 6 : Stereographic projection of joint poles of Sedimentary benches

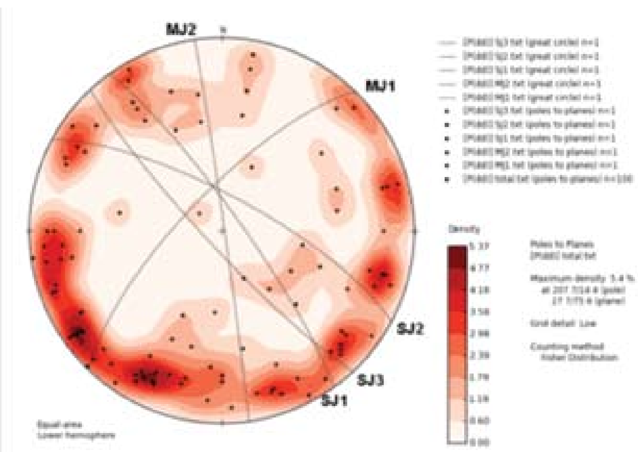

Fig. 7 : Stereographic projection of joint poles of Total Metamorphic and Sedimentary benches. (MJ1= Metamorphic J1,MJ2=Metamorphic J2,SJ1=Sedimentary J1,SJ2=Sedimentary J2,SJ3= Sedimentary J3)

3.2.2 *Folding*

Besides the faults, strata was subjected to Roll-over structure (?) in the SE part of the block as evidenced by Incrop of coal seams and exposures in the quarry of the mine (Photo 3). Part limb of Roll-over structure is faulted in SE part of the block as evidenced by Incrop of I seam. Based on the Incrops of II and IIIA seam, the axis is derived to be in N75^{o}E, since these two seams in the Roll-over structure area are free from faults. However, Plunge of the Roll-over structure could be derived only from the floor contour of IIIA seam as the strata between Incrop of IIIA to Incrop of II seam are intact in the sense, free from faults. The plunge of the Roll-over structure is derived to be 7^{o} based on BH data of BH.NO's- 817 and 819. Similar structure is observed in RG OC-I block located in NE extension of RG OC-II block. However, it is observed from SB-III block onwards, located in the SE extension of OC-II, normal sequence of formations without any such structure.

During normal faulting, in order to prevent formation of void space, the segment of the hanging wall in the neighbourhood of the fault bends down towards the fault with respect to the segments away from the fault giving rise to the typical reverse drag or Roll-over structure also called Roll-over-anticline. Development of Roll-over structure is especially favoured by displacement along listric normal faults (where the dip of the fault plane decreases with depth) with successive blocks showing higher throw towards the direction of downthrow.

Fig. 8A

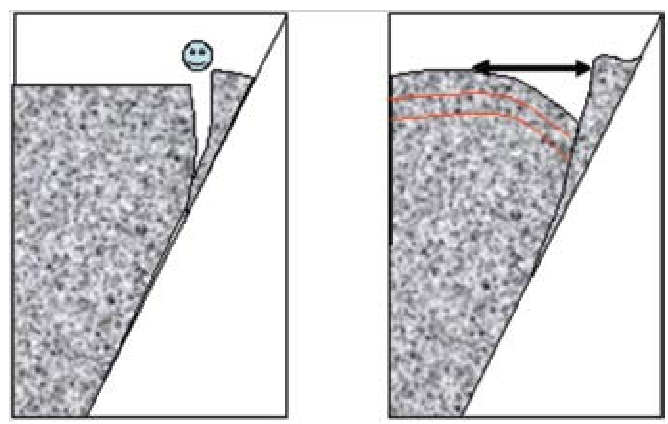

Fig. 8B

Figs. 8 : A & B. Schematic diagram - showing the development of Roll-over structure

Keeping in view of major structural elements viz. Roll-over structure, several faults, variation in the joint pattern is obvious within the block. With an integrated approach. Joints in different parts of the quarry, using different techniques, were studied and the details are discussed in the following pages.

3.2.3 *Faults*

As discussed in para 2.2 Structure, the southern side of RG OC-II, major fault F1 forms the limit of the block (Photo 4). Along this fault, older formations viz. Pakhal Fm, Sullavai Fm and Talchir Fm, were faulted and the Basement rocks of Archaen group came in contact directly with basal Barakar Fm. It is presumed that throw of the fault could be >300m?. Apart from the major fault F1, the block is intersected by several normal faults with varying throw amount and are abutting the major fault F1. Most of these faults were exposed in the quarry during the process of mining. Locations of these faults were surveyed. During the course of present mapping, some of the faults are located within the Metamorphic benches, between Metamorhic & Sedimentary benches and within the Sedimentary benches. Along these fault planes, fault gouge between Metamorphic and Sedimentary benches, displacement within the Metamorphic rocks, drag of the strata within the Sedimentary benches (Photo 5) are recorded.

Reference to"Structural Geology" by De Sitter. (1956), *"A process of geological deformation is no laboratory experiment, and we can try to read the direction of the deformative stress only from the orientation of fold or fault structure. From a general point of view the problem is not difficult to solve. One of the principle stresses is always assumed to be necessarily vertical, because the surface of the earth with gravity perpendicular to its by far the most important plane of discontinuity that enters into the stress field. In the horizontal plane, we can deteremine the largest principle stress as either bisecting the acute angle of a set tear-faults, or, better still, as perpendicular to the longer axis of folding or to the strike of the thrust planes."*

Similar failures of benches were recorded earlier in RG OC-I and GK OC, SCCL. Based on the studies conducted by Basavachary, M., Babu Rao, Y. S. and Dr. Sharma, D. N (1996), it was concluded that

- The failures occurred in the vicinity of major boundary fault where the Gondwana formations abut against metamorphic.
- Clay formations and the presence of water have also affected the slope stability.
- There is an influence of joints, fractures and bedding planes, too.
- From these findings it was suggested:
  - A sufficient barrier is required to be left from the major boundary fault.
  - The pattern of faults, joints, fractures etc. are to be considered in planning blasthole drills and restricting the movement of heavy vehicles.

4. 3D LASSER SCANNING

It is observed from the structural mapping, Scanning data and Geological map that one "Major Fracture-1" trending in N170W developed from surface and extending into Metamorphic benches resulted into subsidence and finally lead to failure (Photo 6). MJ2 joints are making about 12° to this Fracture 1. Further the reasons are, the fracture is at about 60 m from the Incrop of I Seam and is at about 260 m from the Fault F-1. Another "Major Fracture 2" is trending in N53°W and is sub-parallel to major boundary fault F1 which is located at about 70 m on the southern side.

Photo 1 : Typical Joint pattern exposed within sandstone bench showing- "Joint trend", "Joint surface", "Joint aperture" and "Joint spacing."

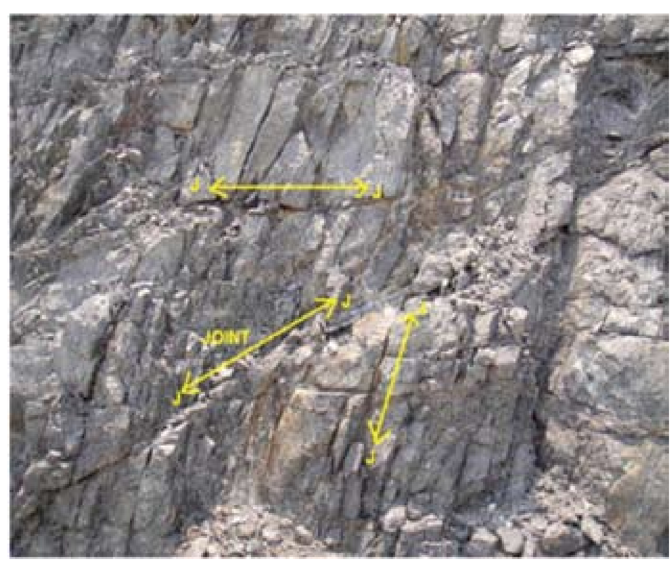

Photo 2 : Well developed Joint system in the Metamorphic rocks.

Photo 3 : A panoramic view of folding in the strata

Photo 4 : A close view of major fault F1 exposed in the quarry where sedimentary benches abut against metamorphic benches

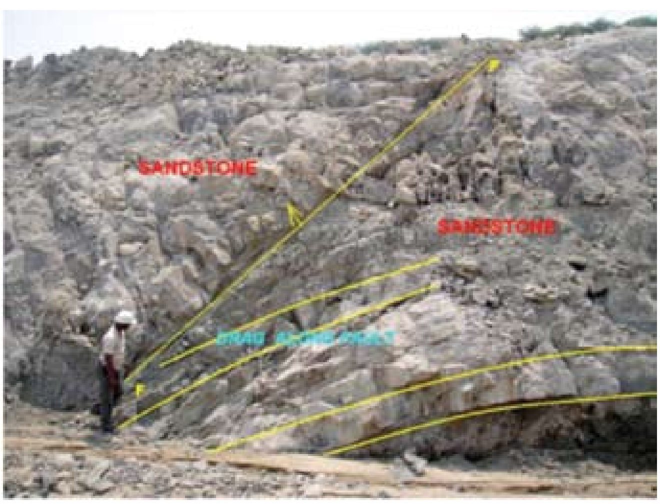

Photo 5 : Exposure of fault within the Sedimentary bench and drag effect of strata on upthrow side of fault

5. SUMMARY AND CONCLUSIONS

1. Mapping was restricted due to (i) inaccessibility, (ii) mostly sedimentary benches are not clear as they are either covered by rolled material or disturbed, etc. Hence, the data was generated from the locations where-ever prominent structural features are observed. Filtered the data during mapping at field itself, without compromising with the conditions.

Photo 6 : Displacement of bench due to major fractures showing original position and after subsidence.

2. Based on the mapping carried out in the present study area, the findings are concluded as follows:
   - JOINTS: In the Metamorphic benches, only two most prominent sets of Joints are found wherein J1 trends in N35°E and J2 trends in N5°E. Whereas in Sedimentary benches, three sets of Joints viz. J1 in N75°W, J2 in N65°E and J3 in N35°W.
   - ROLL-OVER ANTICLINE: Roll-Over Anticline structure in the study area with a fold axis of N75° E and a plunge of 7° is responsible for variation in the trends of Joints.
   - FAULTS: In the study area, including Major Fault F1 (between Metamorphic and Sedimentary benches), faults within the Metamorphic benches as well as within Sedimentary benches are mapped and their impact on stability of benches viz. wide cracks, failure etc. are reported. It is inferred that the variation in the Joint trend is influenced in the vicinity of faults.
3. In view of inaccessibility in generating the desired data, SIROVISION is found to be useful.

4. Further, it is found that the trends of Joints are controlled by Structural elements viz. Roll-over structure, folds etc and are localized.
5. Failure of benches are found to be result of unfavorable orientation of discontinuities viz. Joints,faults etc.
6. Kinematic analysis of the available data will be more helpful to attribute the reasons for failure of benches and it's remedial measures.

6. ACKNOWLEDGEMENTS

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BIOGRAPHICAL DETAILS OF THE AUTHORS

Dr.D.N. Sharma, obtained M.Sc. (Tech) in Applied Geology from Andhra University, Visakhapatnam. Awarded Ph.D in Engineering Geology by the Indian School of Mines, Dhanbad. Working in the Singareni Collieries Company Limited since +32 years in Coal Exploration and specialisation in Geotechnical studies. Visited China for "Longwall Mechanisation". About fifty technical papers were published in International & Indian Journals, Proceedings of International and Indian Seminars/Conferences/Workshops, etc. Presented a documentary on "GEO-ENGINEERING STUDIES IN COAL MINES" in the National channel of Doordarshan through UGC-EMRC-CIEFL, Hyderabad. Associated with several International Experts in different Projects of SCCL. Life member of "The Geological Mining and Metallurgical Society of India" Kolkatta, "Indian Society of Engineering Geology", Lucknow and "Indian Society for Rock Mechanics & Tunneling Technology" New Delhi.

R.Vinay Kumar, post graduated in Applied Geology from the Osmania University, Hyderabad in 2008. In the same year he joined in National Geophysical Research Institute (NGRI), Hyderabad as Project fellow in Paleomagnetic division and worked for one year. Subsequently he joined in Singareni collieries Company as a geologist in 2009.He is having work experience in Geophysical methods of coal Exploration viz.Well logging, interpretation of data etc. from 2009 to 2011. Since 2011 he is looking after the Geological aspects of coal exploration and Geotechnical studies of Underground and opencast mines.

G.Rajendar, post graduated in Applied Geology and Hydrogeology from the Osmania University, Hyderabad in 2002 and 2005 respectively. In 2009 he joined Singareni collieries Company as geologist and he is involved in coal exploration and hydro geological investigations such as identification of feasible locations for construction of water bore wells ,monitored seasonal phreatic and piezometric water levels, Environmental studies, hydrogeological environ of South Godavari mining lease etc. from 2009 to 2013. Since 2013, he is looking after the Geological aspects of Coal exploration and Geotechnical studies of both Underground and opencast mines.

FATIGUE PROPERTIES OF INTACT SANDSTONE IN PRE AND POST-FAILURE AND ITS IMPLICATION TO VIBRATORY ROCK CUTTING

M.N. Bagde
CSIR-Central Institute of Mining & Fuel Research, Regional Center, Nagpur, India

S. Karekal
Centre for Earth Science and Resource Engineering, CSIRO, Australia

ABSTRACT

The pre- and post- failure fatigue properties of intact sandstone subjected to uni-axial cyclical loading in the laboratory is presented with its possible implication to vibratory rock cutting. The fatigue results subjected to sinusoidal, ramp and square waveforms at cyclic loading frequency of 5 Hz and peak amplitude of 0.05 mm is discussed herewith. It is observed that fatigue behaviour is a function of the dynamic energy of the load and the shape of the waveform. From the presented results, the practical significance of the behaviour of rock and rock masses within the excavation systems subjected to cyclic loads, especially in the vibratory rock cutting is put forward. To substantiate the cyclic breaking of rock under vibratory loading condition, a numerical simulation results using discrete element code is presented. Two loading cases were considered. In the first case, the wedge pick was loaded non-cyclically (monotonically), and in the second case it was loaded dynamically under cyclic sinusoidal loading at 50 Hz frequency. A load-deformation curve under monotonic (non-cyclic) loading condition and cyclic loading conditions were examined. The model showed a drop of about 25% in peak strength in the case of cyclically loaded wedge pick compared to monotonic or quasi-static loading of the wedge pick cutting. It is inferred that the vibratory loading has benefits in fracturing rock at relatively lower load compared to conventional loading.

1. INTRODUCTION

The understanding how dynamic loading influence rock failure and its behavior has a great significance in excavation systems prone to extreme rock-burst loading, drilling, crushing, blasting and vibratory rock cutting etc.; however, there is almost rare literature on this. It is well reported in the literature that intact and failed models of jointed rock are extremely susceptible to cyclic fatigue failure. For the detailed review and studies on the subject refer to various publications[1,2,3,4,5,6]. In cited various publications, fatigue behaviour of sandstone, siltstone and conglomerate rocks obtained from the rock-burst prone Ostrava-Karvina coalfield is presented. The significance of low-load frequency and high amplitude in fracturing, micro-fracturing and degradation of various rock properties in cyclic loading is discussed in those publications. The results presented therein were corroborated with the rock burst problem and mitigation of the same. However, here the findings reported earlier were attempted to corroborate with its implications to vibratory rock cutting. As discussed in one of the paper[4] fracture and fragmentaion of sandstone rock was found to affect significantly at low loading frequency when compared to higher frequency and it was suggested that the most efficient method of rock cutting, excavation, fragmentaion and crushing could be low-speed loading. The effcet of loading waveform on fatigue properties of the intact rock in pre-failure state are discussed in[5], while, post-failure properties are reported in[6] and discussed with relation to seismic rock burst conditions. However, in the present paper, the reported earlier findings will be related to vibratory rock cutting with its significance and importance in rock cutting. Here, an attempt has been made to corroboarte fatigue behaviour of the rock with its applications to vibratory rock cutting, its significance and importance in rock cutting with results obtained from the numerical simulation using discrete element modelling.

2. TEST EQUIPMENT AND SCHEME

The deatils of the tested rock, equipment and test scheme in brief is provided in the following. For the detailed explanation refer to[5,6]. The rock samples were obtained from the rock burst prone Darkov coal mine in the Ostarva-Karvina coal basin in the Czech Republic. The samples were of 1:1 diameter to length ratio with average diameter of 47.5 mm. Samples were prepared and tested according to ISRM testing procedure and guidelines[7].

The testing equipment was MTS-816 rock test system. The details about the rock types, testing equipment, test scheme and waveform types and their characteristics can be found elsewhere[5,6]. The tests were conducted with axial displacement controlling loading system and the dynamic load was specified as a sine, ramp and square cyclic compressive respectively for a given set of test conditions. The illustration of different nomenclatures associated with cyclic loading is provided elsewhere[5,6]. The illustration of real loading condition on time-displacement curve throughout the uniaxial cyclic loading test is given in Fig. 1.

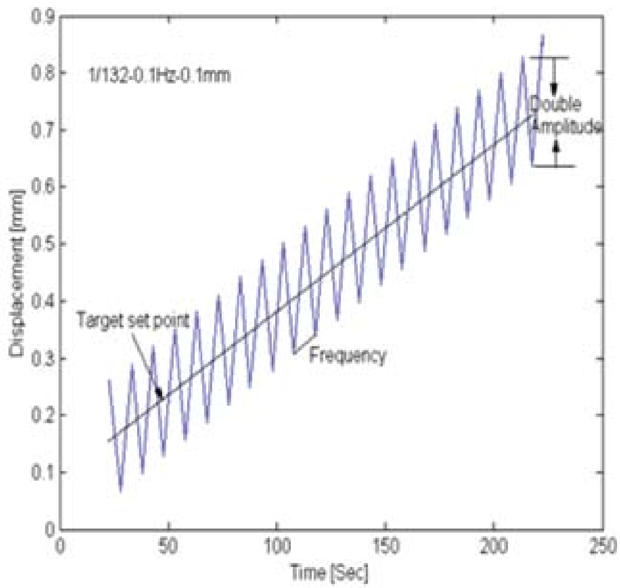

Fig. 1 : Time-displacement curve illustrating uniaxial dynamic cyclic loading condition (example for 0.1 Hz frequency and 0.1 mm amplitude)

The shape of a waveform determines the loading/unloading rate, the rate of change in the loading/unloading rate, and the residence period at the peak stress. The illustration of three waveforms with their charactertistics cab be found in detailed in[5,6]. It is well reported and established in rock mechanics that loading rate strongly influences the rock behaviour. The numerous studies is being reported in the literature where strain rate or loading rate effect on various rock properties is discussed. The detailed discussion about the maximum loading rate in a waveform and its influence on fatigue rock behaviour in uniaxial cyclic dynamic loading conditions is discussed in the following with their possible applications to vibratory rock cutting.

3. EVALUATION OF ROCK PROPERTIES UNDER CYCLIC LOADING

The data from the cyclic loading tests were analyzed to obtain various fatigue properties of the tested rock. An illustration of stress and modulus computation from peak-valley data using a computer programme is given in Fig. 2. For the detailed explanation refer to[5,6].

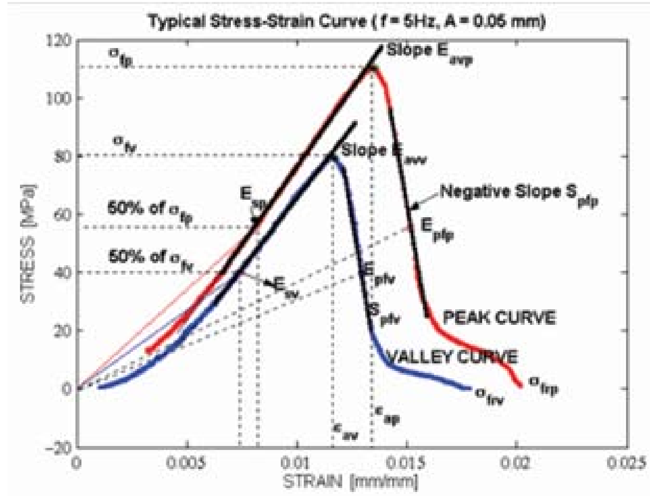

Fig. 2 : Typical stress-strain curve with illustrating calculation of various rock properties in pre- and post-failure curves from peak-valley data in uni-axial cyclic loading condition

4. EXPERIMENTAL RESULTS AND DISCUSSION

Tests were conducted to study the effect of waveform on the cyclic fatigue behaviour of sandstone rock in uni-axial cyclic loading conditions. The sinusoidal, ramp and square waveforms were used with a load frequency of 5 Hz and amplitude of 0.05 mm. The results presented and discussion made herewith are based on the average results. On an average three samples were tested for individual testing conditions.

The dynamic fatigue strength obtained was higher in the case of ramp loading waveform compared to that of sinusoidal and square (Fig. 3). The post-failure negative slope (Fig. 4) found to be higher in the case of ramp, followed by sinusiodal and least in the case of sqaure waveform. The least post-failure negative slope in the case of sqaure waveform suggests that under such condition rock would fail in more brittle and or voilent manner due to the residence period (means constant load amplitude during residence period) and high loading rate when compared to others.

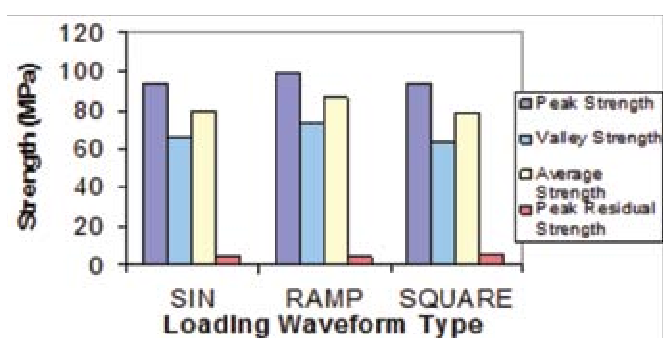

Fig. 3 : Strength with loading waveforms[6]

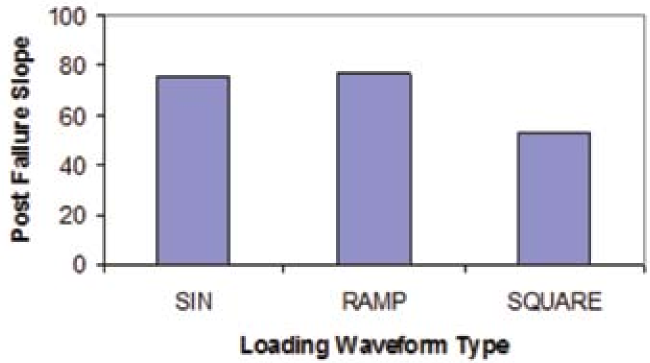

Fig. 4 : Post failure slope with loading waveforms[6]

Also, when compared results presented in Fig. 5, the average dynamic Young´s modulus (Eavd) was higher in the case of ramp than other two waveforms considered. In the case of sinusoidal, it was found to be least. In the case of secant modulus, it was least in the square and higher in the case of the sinusoidal waveform. Also, reversible modulus was found to be higher in the case of ramp, followed by sinusoidal and least in the case of square waveforms. The post-failure modulus found to be the least in the case of square waveform. When compared all those presented results as above, it is concluded that the damage accumulated most rapidly under the square waveforms compared to others. The maximum loading rate in a waveform found to strongly influence the damage accumulation in rock and it is the square waveform-the most damaging one from the presented results. Also, the larger disparity between the secant and the Young's modulus values suggest the greater initial crack density in the case of square waveform. From the presented results, it is concluded that residence period and thus constant loading amplitude during residence time in the case of square waveform compared to others helps in better fragmentaion and micro-fracturing of the rock resulting in degradation of the most of the fatigue rock properties.

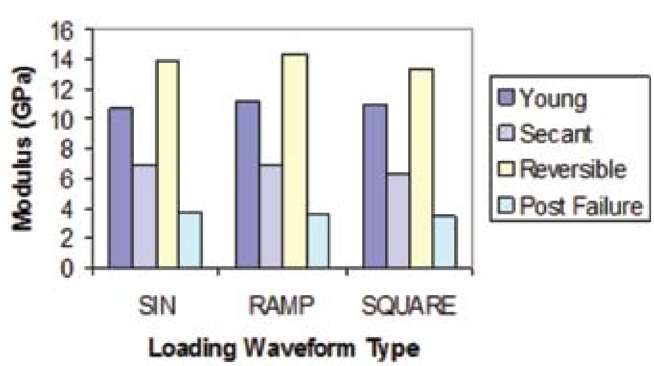

Fig. 5 : Modulus with loading waveforms[6]

The dynamic energy (D_e) utilized to cause rock failure was found least in the case of ramp waveform compared to that sinusoidal and square (Fig.6). In the case of square waveforms, it was found that dynamic energy requirement was more to cause failure of the rock samples. From this, it could be concluded that a ramp waveform is the least damaging of those considered. The damage accumulated most rapidly under sqaure waveforms with a high dynamic energy requirement and followed by sinusoidal and ramp waveforms. These results demonstrate that it is the loading rate or loading waveform that mainly contributes to the damage in rock specimens. Thus, it is inferred that the higher the workdone per cycle during low cycle fatigue tests, the shorter the time to failure will be. It means that the higher amplitude and or constant amplitude for long-residence period and low-frequency is of the great significance in cyclic loading conditions which will help in better rock fragmentaion and fracturing and early failure means low-fatigue life. It is found that crack propagation is better allowed to occur in the case of low-loading frequency than the higher one. The loading amplitude is found to have significant influence on the rock fatigue behaviour and degardation of the rock properties.

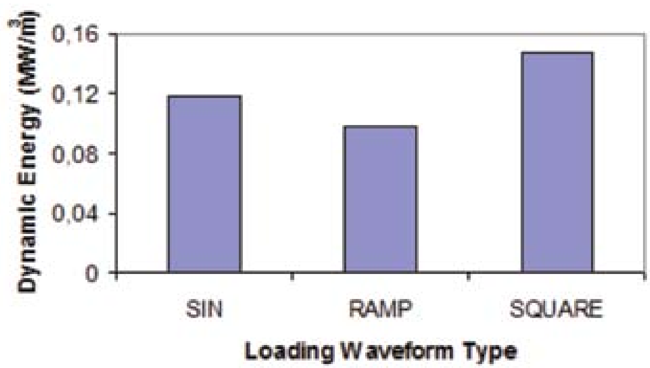

Fig. 6 : Dynamic energy with loading waveforms[5]

This study focused on investigating the effects that waveform and amplitude have on the dynamic cyclic fatigue behaviour of intact sandstone rock. Based on the work presented, the following inferences are made:

Fatigue behaviour of the rock due to uniaxial cyclic compression is a function of the shape of waveform and the workdone by the load. The damage accumulates most rapidly under the sqaure waveforms with a high energy requirement. The ramp waveform is less damaging than either square or sinusoidal waveforms. The ramp waveform found to be uniform in loading and unloading with less dynamic energy requirement to cause failure in the rock samples for a given loading frequency and amplitude. The loading waveforms strongly influences the damage accumulation (accumulated deformation) under cyclic loading conditions. It is found that the maximum loading rate in a waveform strongly influences the damage acumulation in rock. It is found that type of loading waveform affects the various pre- and post-failure rock properties presented here in dynamic uniaxial cyclic loading conditions. The possible applications of the presented fatigue behvaiour results is discussed with reference to vibratory rock cutting and corroborated with the findings form the discrete element simulation results in the following.

5. DISCRETE ELEMENT SIMULATION OF VIBRATORY ROCK CUTTING

To substantiate the new technique of breaking hard rock under variable amplitude cyclic loading and unloading (or under vibratory loading condition), a numerical simulation using discrete element code (Universal Distinct Element Code, UDEC) was used.

The numerical methods are becoming more attractive and popular now days for modeling the complex rock-tool interaction processes because idealized analytical methods with simplified assumptions do not fully capture the failure process. The various continuum numerical codes such as FEM, BEM and FLAC have been used to understand the rock-tool interaction process. These models have been developed either through incorporation of failure theories or through the use of fracture mechanics with pre-existing cracks[8,9,10]. The Discrete Element Method (DEM), particularly Particle Flow Code (PFC), has been used to simulate the failure process of rock by mechanical indentation[11].

The general process of wedge cutting using discrete element code is illustrated in Fig.7.

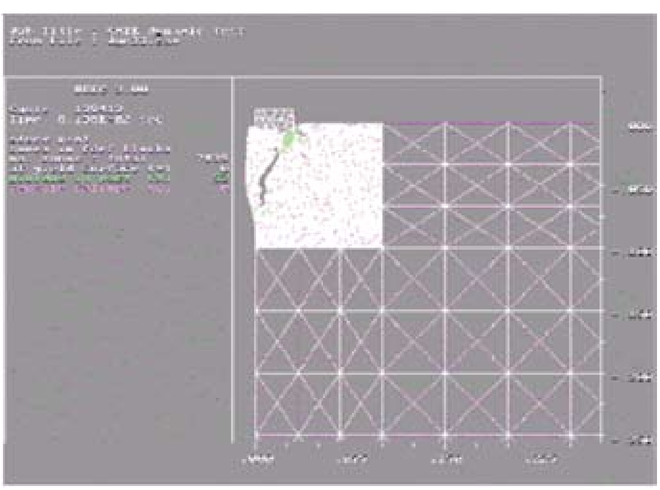

Fig. 7 : Crushing and fracturing in wedge pick cutting of simulated material[12]

When a pick is pressed against the rock at a certain edge distance, a plastic zone (crushing) forms beneath the indenter, and a distinct crack emanates from the crushed zone and extends laterally to its free surface forming a chip (Fig.7). The DEM seems to capture both the crushing and crack development processes observed in experimental rock cutting.

In the simulations carried out[12], two loading cases were considered. In the first case, the wedge indenter was loaded non-cyclically (monotonically), and in the second case was loaded under variable amplitude at 50 Hz frequency (Fig. 8).

The rock block having dimensions of 1m depth and 1m width was used for the simulation. A square of 100 mm by 100 mm of the inner region (Fig. 7) was further subdivided into smaller jointed blocks to treat this region as a discontinuum, and to understand the fracturing process better[12]. Viscous boundary conditions were applied along the left boundary to prevent failure due to the reflection of a wave from the boundary. A constant velocity boundary condition of 100 mm/s was applied along the top of the wedge pick (under quasi-static loading). In another case, in order to investigate the effect of vibratory motion of the cutter on the force-penetration curve, the cutter/indenter was vibrated at 50 Hz, with amplitude of 1.5 mm. In all the cases, the cutter was allowed to move in Y – direction (vertically down) and its movement in X direction was restricted.

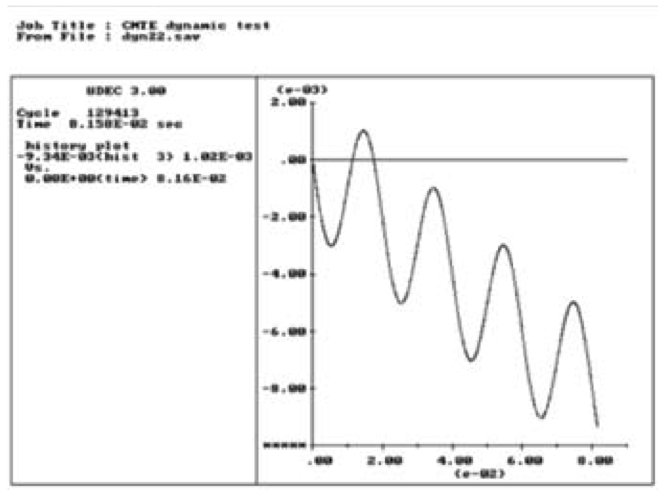

Fig. 8 : Variable amplitude cyclic displacement boundary condition with time

The Mohr-Coulomb failure criterion with a strain softening constitutive model was adopted for deformable blocks. For joints, a Coulomb joint slip failure constitutive model was chosen as it allows understanding of the general indention mechanisms. For comparison of force-penetration curves in cyclic and non-cyclic motion of the cutter, a continuously yielding joint model was adopted[12]. In all the cases, a friction angle of 10o was chosen between the indenter and the rock. Also the material properties were kept same for all the simulations. Figs. 9 and 10 show the force-penetration curves under these loading conditions.

These simulation results[12] show that a peak force of 13.2 kN was attained before failure was achieved during monotonic (non-cyclic) loading. However, with cyclic loading the peak force achieved was reduced to 9.93 kN (about a 25% drop). It can be inferred from the simulation results that variable amplitude cyclic loading requires relatively less force to fracture compared to non-cyclic loading in the configuration considered. From the studies where fatigue behavior of the rock is discussed in cyclic loading also inferred that low-loading frequency with higher amplitude or constant loading amplitude during residence period in the case of square waveform was found to be most damaging. It is also observed that crack propagation will be better in the case of low-loading frequency and damage accumulation will be more in the case of higher amplitude.

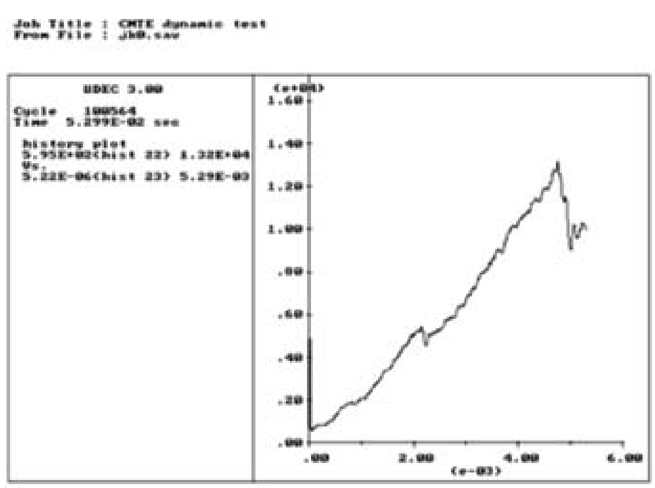

Fig. 9 : Load-Displacement curve in non-cyclic loading

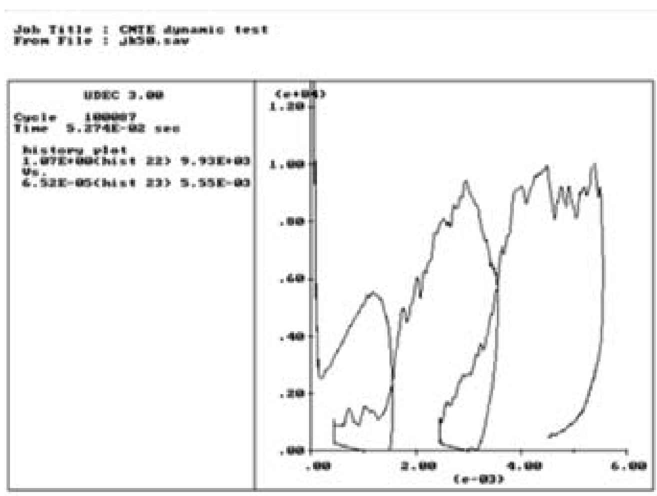

Fig. 10 : Load-Displacement curve in variable amplitude cyclic loading

Further work is needed before findings of the present study can be applied in a quantitative sense, and more focus is needed on the experimental program on the vibratory rock cutting and verification through model simulation with appropriate constitutive equations.

ACKNOWLEDGEMENTS

The views expressed in this paper are of those authors and not necessarily of the institute, to which they belong.

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BIOGRAPHICAL DETAILS OF THE AUTHORS

Dr. Manoj Namdeo Bagde graduated in Mining Engineering with first division in the year 1993 from Visvesvaraya Regional College of Engineering, Nagpur (presently known as VNIT). He has obtained his M. Tech in Rock Mechanics in the year 1994 from Indian Institute of Technology, Delhi. He was awarded Ph. D. degree in Mining Geo-mechanics in June 2004 by VSB-Technical University, Ostrava, Czech Republic under Czech Govt. Scholarship. He has worked as a visiting fellow during 2010- 2011 at McGill University, Canada under BOYSCAST Fellowship of DST, GOI. He has joined Central Institute of Mining and Fuel Research in the year 1996 and presently working as Principal Scientist in the area of Rock Mechanics and Mining Engineering at CIMFR Regional Center, Nagpur. His area of interest includes Rock Mechanics and Ground Control, Mining Methods and Ground Support in Mines, Tunnels and Caverns, Backfill failure in hard rock metal mines etc. He has made a significant contribution by publishing technical R & D articles in well known Journals of International repute. Till date he has to his credit 7 International Publications, 6 in National Journals and more than 40 publications in International & National Conferences and Seminars etc. He has visited Australia, Canada, Hong Kong and Czech Republic and presented technical papers at different worldwide forums.

Dr. Shivakumar Karekal (Project Leader, CSIRO, Australia) has Bachelor and Master degrees in Mining Engineering and PhD in Mining, Mineral and Materials from the University of Queensland. He has 23 years of research experience worked at several research organizations such as National Institute of Rock Mechanics (India), Centre for Mining Technology and Equipment (CMTE), Australia, Lecturer, Division of Mining, Mineral and Materials, University of Queensland, and CRC Mining. He was associated with world renowned research organizations such as Norwegian Geotechnical Institute (NGI), Norway, National Institute of Advanced Industrial Science and Technology (AIST), Japan, Sustainable Mining Institute (SMI) and JKMRC, Australia. He was a nodal coordinator for Mining Education Australia (MEA) geo-mechanics courses and a national coordinator for drilling and blasting curriculum for Mining Education Australia (MEA) administered by the Mineral Council of Australia (MCA). He is currently with CSIRO. He has executed several national and international projects and has patents in advanced cutting technology in USA, Germany, South Africa, and Australia. His main interest is the application of advanced and innovative technologies for solving mining industry problems.

DESIGN OF LINING FOR A MINE SHAFT AND DECLINE USING NUMERICAL MODELLING TECHNIQUES

Islavath Sreenivasa Rao
Department of Mining Engineering, University of Engineering (KU), Kothagudem, India

Debasis Deb and Sujeet Bharti
Department of Mining Engineering, IIT Kharagpur, India

ABSTRACT

A chromite ore body located in eastern India is to be extracted by sub-level open stoping method. A vertical circular shaft and a decline are required to be developed for transporting man, material and air. It is decided that the shaft and the portal of the decline will be located in the footwall side of the ore body. The top most cross-cut from the decline will contact the ore body at about 170 mRL. This study has been conducted to evaluate the engineering properties of lining material and its thickness for the proposed shaft and decline geometry. In order to perform such tasks, borehole rock samples are tested to estimate the engineering parameters of the country rocks and ore body. Apart from that rock mass rating (RMR) and geological strength index (GSI) of rocks are also determined. Two dimensional numerical modelling techniques have been employed to estimate the stress and displacement distributions around the decline and shaft. This paper presents the methodology and results of this study and demonstrates the efficacy of numerical modelling technique in stability analysis of shaft and decline.

INTRODUCTION

Out of total reserves in India, 95% of chromite occurs in Orissa, and it has wide usage in chemical and metallurgical industries. The chromite deposit in study extends 290 m in strike length with an average width of 20 m. The top portion of the deposit is located about 30 m below the surface and extends vertically upto 200 m. The dip angle of the ore body is 100 degree with the horizontal or 800 degree with the vertical. Country rocks of chromite deposit are weathered serpentinite, serpentinite, quartzite and pyroxenite. A quartzite hill exists on the surface towards the dip side of the ore body. The bottom most RL of the surface is 210 mRL.

It is proposed that the chromite ore body will be extracted by sub-level open stoping method with late filling, if required. For this purpose, a vertical circular shaft and a decline need to be developed to access the ore body. The shaft will be located in the footwall side of the ore body. The portal of the decline will also be located in the footwall side of the ore body. The top most cross-cut from the decline will contact the ore body at 170 mRL. This study is required to be conducted to determine suitable thickness of lining of mine shaft and decline and safe location of the shaft. In order to perform such tasks, borehole rock samples have been tested to estimate the engineering parameters of the country rocks and ore body. Rock Mass Rating (RMR) and Geological Strength Index (GSI) have also been estimated for the same. Two dimensional numerical modelling techniques have been employed to estimate the stress and displacement distributions in the pillars, walls of the stopes, decline and shaft.

OBJECTIVE OF THE STUDY

This study has two main objectives:

(i) Determination of geotechnical and rock parameters of hanging wall, footwall and ore body of the chromite deposit to use as input to the numerical modelling software.

(ii) Design and stability analysis of the mine shaft and decline using numerical modelling techniques. The method of stoping is ‘sub-level open stoping method with late filling, if required’.

GEOTECHNICAL STUDY AND LABORATORY TESTS OF ROCKS

Rock Quality Designation (RQD), Rock Mass Rating (RMR) and Density of rocks (γ)

RQD, RMR and density of ore body and the country rock is determined. RQD varies from 56.81 to 83.65 as listed in Table 1.

Table 1 : RQD RMR of different type of rocks

Rock Type	W.Serpentinite	Serpentinite	Chromite	Pyroxenite	Quartzite
RQD (%)	56.81	83.65	75.41	78.37	63.66
RMR	47	71	75	74	76
Density (kg/m^3)	2745	2745	3537.9	2350	3000

RMR of weathered serpentinite is about 47 and that of all other rocks are above 70. Hence, rocks are found to be competent at greater depth.

Mechanical Parameters of Core Samples

The rock core samples collected from the mine site were cut and polished to the size recommended by the International Society of Rock Mechanics (ISRM). The length to diameter ratio of the uni-axial compressive strength and the tensile strength are 2.5 to 3.0 and 0.5 respectively. The UCS, tensile strength, modulus of elasticity and Poisson's ratio of the rock samples are determined in the laboratory to use as input to the numerical model and are listed in Tables 2 and 3.

Figures 1a and 1b show the sample under uni-axial compression test and tension test. Figure 1c shows the stress-strain behaviour of the chromite samples tested in the laboratory. These figures depict both axial and lateral strains with load increments. It may be noted that the axial strain is considered to be compressive (+ve) while lateral strains are tensile (-ve).

The values obtained from the test are an estimate of the 'Intact Rock Material', whereas, the larger rock mass in situ contains local cracks, fractures and discontinuities, which reduce the strength of the rock mass considerably. Hence, to incorporate this reduction in the rock strength, the rock properties were modified, according to the following equations (Marinos and Hoek, 2000):

$$\sigma_{cm} = (0.003 m_i^{0.8})\sigma_{ci}(1.029 + 0.025 e^{-0.1 m_i})^{GSI} \qquad ...(1)$$

It is found that elastic modulus mostly varies linearly with UCS and hence, deformation modulus is also determined using equation (1) by replacing UCS by elastic modulus of intact rock.

Again, considering the unforeseen events such as water conditions, local joints and fractures, elastic modulus of rock mass (E_c) is further reduced to 80% as

$$E_{cm} = 0.8E_c \qquad ...(2)$$

Here, m_i = the Hoek-Brown parameter of intact rock, σ_{ci} = uniaxial compressive strength (Intact rock core), σ_{cm} = uniaxial compressive strength (rock mass), E_{ci} = elastic modulus (Intact rock core), E_c = elastic modulus (rock mass), and E_{cm} = elastic modulus (reduced by 80%). In this study, Poisson's ratio (ν) of intact rock and that of rock mass is kept the same.

NUMERICAL MODELLING FOR DESIGN OF THE MINE SHAFT AND DECLINE

The Finite Element Method

Finite element method is an effective tool for the analysis of mechanical and structural components of machinery. This method is amenable to systematic computer programming and offers a scope for application to a wide range of problems for analysis. This method is now adopted in almost all branches of engineering where complex structures, fluid dynamics problems, mine and tunnel structures and similar problems are addressed (Bathe, K. J., 2007).

The basic concept in this approach is that a body or structure can be divided into a finite number of smaller units of finite dimensions called 'elements'. The original body or structure is then considered as an assemblage of these elements connected at finite number of joints called

(a) Uni-axial compression test (b) Tension test (c) Stress-strain relationship curve

Fig. 1 : Testing of mechanical properties of rocks

Table 2 : UCS and Tenile strength of different rocks

Rock type	W. Serpentinite	Serpentinite	Chromite	Pyroxenite	Quartzite
Avg. UCS (MPa)	132	152.56	113.18	115	110
Avg. (MPa)	4.05	11.82	10.33	3.2	3.5

'nodes' or 'nodal points'. The properties of these elements are formulated and combined to obtain the solution for the entire body or structure. The global system of equations is developed as follows (Deb, 2010):

$$\{F\} = [K]\{q\} \quad ...(3)$$

Where, $\{F\}$ = global force vector, i.e. forces in each node,

$[K]$ = global stiffness matrix based on material properties,

$\{q\}$ = displacement vector containing each node.

Necessary (essential) boundary conditions in terms of displacements at some selected nodes are applied before solving this system of equations.

Two Dimensional Model

Two 2D-finite element models have been developed to analyze the stress distribution around the decline, the stope and the shaft area and also have been estimated the plastic zone around these three openings. The primary motivation of analyzing in 2D is to determine the failure zone around the decline and the shaft. All 2D models are analysed considering Drucker-Prager materials in plane strain conditions. It means that rock mass is allowed to yield based on its strength and the developed plastic strain intensity has been observed around the excavations (Deb De tal., 2008). The 2D models are described as follows:

(I) Vertical section comprises host rock, ore body, drives and the decline.

(II) Horizontal section comprises host rock, ore body and the shaft.

Finite Element Models - 2D (Vertical Section)

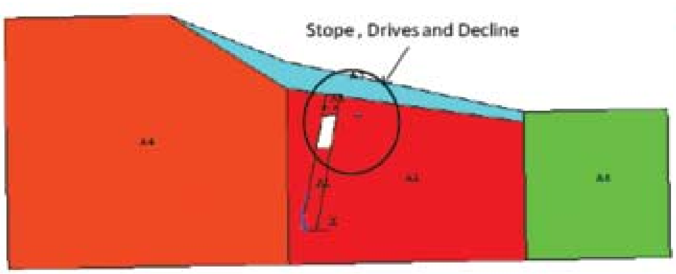
The vertical section of the 2D model is comprised of host rock, ore body and the decline. Vertical section is taken at 165 m away from the boundary of the adjacent mine. Figures 2a and 2b show the excavation model of vertical section which consists of stope, decline and surrounding rock. The in-situ model consists of in-situ ore body with host rock.

Material Properties

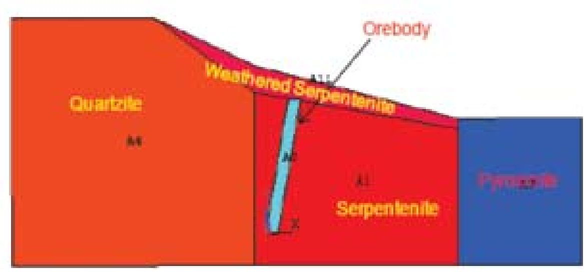
Rock mass properties of these models have been further reduced to simulate more jointed rock mass conditions. Table 3 lists the rock and rock mass properties those are used for finite element modelling for all 2D models. It can be easily seen that compressive strength of rock mass of each rock type is considered at the lower side to incorporate jointed rock mass with few centimetre spacing, weathered and watery conditions.

Generation of Finite Element Meshed Models

The meshing of the excavation model produced an average of around 3,715 six-noded triangular elements and around 7,538 nodes (Figure 3a). Little variations are noted in node counts with in-situ model geometries. Figure 3a shows the meshed model of vertical section of excavation model. In general, finer mesh is developed around the stope, the drives and the decline zones. In those zones, finer mesh is required for better evaluation of displacements, stresses and strains. Coarser mesh is developed in the rock-body away from the mining-affected zones.

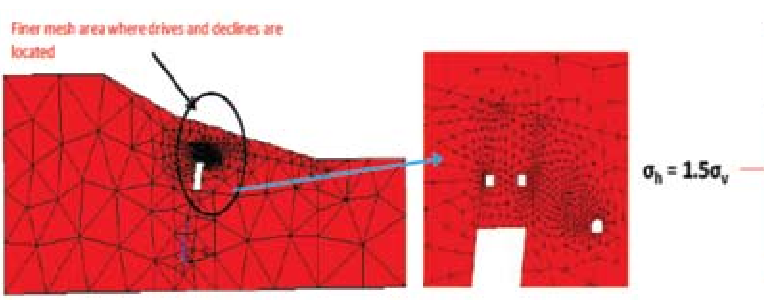


2(a) Vertical section of excavation model

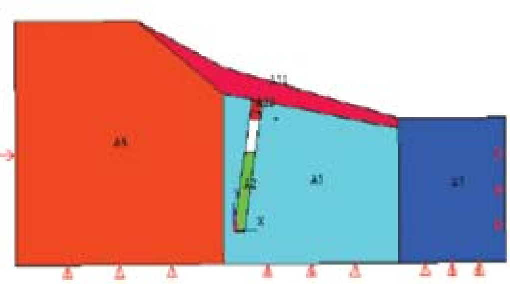


2(b) Vertical section of in-situ model

Fig. : 2a and 2b



3(a) Meshed model



3(b) Boundary conditions are applied

Fig. : 3a and 3b

Table 3 : Material properties used in the models

Rock or Material	Modulus of Elasticity (MPa)	Poisson's Ratio (ν)	Density (kg/m^3)	Compressive Strength of Rock Mass (MPa)	Cohesion of Rock Mass (MPa)	Angle of Friction Rock Mass (Deg)
Weathered Serpentinite	1840.107	0.138	2745.0	11.525	3.00	35
Serpentinite	4064.160	0.138	2745.0	3.924	1.25	25
Chromite	6816.800	0.144	3537.9	3.570	1.25	20
Quartzite	5090.627	0.150	2350.0	17.156	4.00	40
Pyroxenite	1717.700	0.200	3000.0	11.939	4.18	20

Loading and Boundary Conditions

To apply loads, the models are constrained from two sides, those are (i) perpendicular to the strike of ore body and (ii) bottom surface. Surface traction pressure is applied to another side as shown in Figure 3b. The following equation is used for calculation of horizontal stress in finite element model according to depth of the ore body.

$$\sigma_h = 1.5 \times \sigma_v \qquad ...(4)$$

Where, σ_h : Horizontal stress perpendicular to the strike of the ore body (MPa), σ_v : Vertical stress (MPa)

Results and Discussions

All 2D finite element models have been analyzed using elasto-plastic behaviour of rock materials in plain strain condition. Results are enumerated in terms of principal stress distribution and plastic strains around the excavations.

Major Principal Stress (σ_1) Distribution

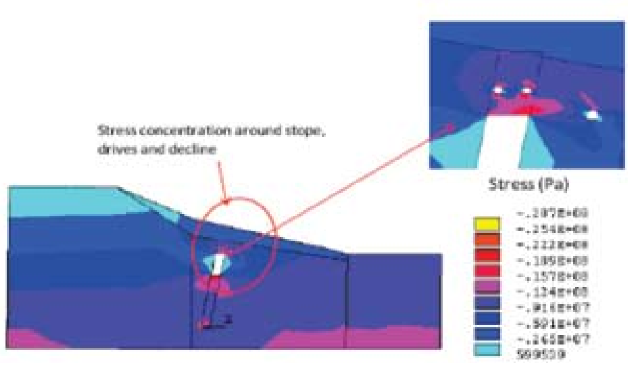
The major principal stress distribution around the stope and the decline is shown in Figure 4a. It is found that high stress concentration occurs at the corners of the stope. The peak major principal stress may range in between 15.7 MPa and 22.2 MPa. Tensile stress may develop in the hanging wall and foot wall rock masses.

Minor Principal Stress (σ_3) Distribution

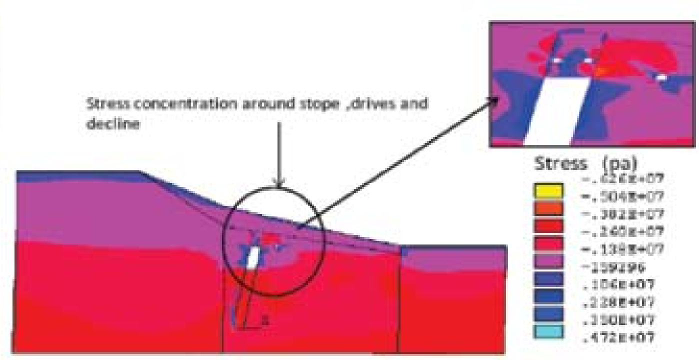
Distribution of minor principal stresses around the stope and the decline is plotted in Figure 4b. Tensile stress develops in the hanging wall, foot wall and the surrounding of the drives and also on the surrounding of the decline. Tensile stress ranges in between 2.28 MPa and 5.04 MPa depending on the location. This phenomena will cause plastic strains (yield zone) around the excavation.

The Plastic Zone

As mentioned earlier, the 2D vertical section has been analyzed to determine the plastic zone in the rock mass surrounding the decline. Figure 5 shows the intensity of plastic strains that has developed around the decline. In this case, a higher value indicates more failure of rock mass. These results clearly signify that proper support system is required to protect decline roof and side walls. The results also imply that a few locations, the crown pillar may yield and hence it requires reinforcement.



4a: Major Principal stresses of vertical section



4b: Minor Principal stresses of vertical section

Fig. : 4a and 4b

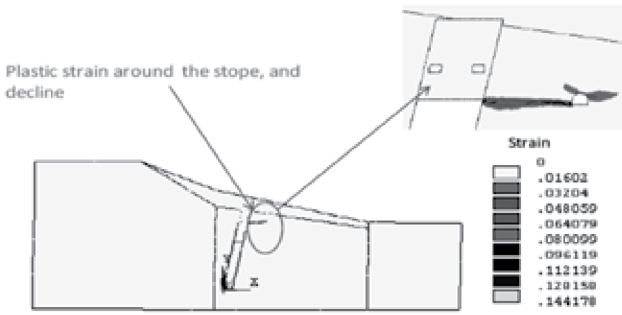


Fig. 5 : Distribution of intensity of plastic strain

Finite Element Models - 2D (Horizontal Section)

Horizontal Section Comprising Host Rock, Orebody and the Shaft

This model is developed to determine the plastic zone around the shaft area. The horizontal section has been taken at 155 mRL. This section is comprised of the shaft, plan section of rib pillars; excavations and host rock mass (Figure 6). It may be noted that authors have no intention to analyze stresses or displacements developed in this model. They are mainly interested to observe the failure zone surrounding the shaft and determine the strength properties of lining material based on that.

Generation of Finite Element Meshed Model

This 2D model has 1,355 six-noded triangular elements and around 2,872 nodes. Figure 6a shows the horizontal section of meshed model of ore-body and its surrounding areas. In general, finer mesh is developed around the stope and shaft zones. Coarser mesh is developed in the rock-body away from the mining-affected zones (zone of no-rock movement due to mining).

Loading and Boundary Conditions

The model is constrained from two sides, (i) along the strike of the ore body, and (ii) perpendicular to the strike of the orebody. Surface traction pressure is applied from other two sides. The following equation is used for calculation of horizontal stress at a depth of 155 mRL.

Horizontal stress along the strike of the ore body,

$$\sigma_u = \sigma_v, \text{ MPa} \quad ...(5)$$

Horizontal stress perpendicular to the strike of the ore body, $\sigma_h = 1.5 \times \sigma_v$, MPa ...(6)

Results and Discussions

The model is analyzed considering Drucker-Prager yield criterion in plane strain conditions. Material properties mentioned in Table 3 are also applied for this model. Results in terms of plastic zone around the shaft area are analyzed and presented below.

Distribution of Plastic Strain

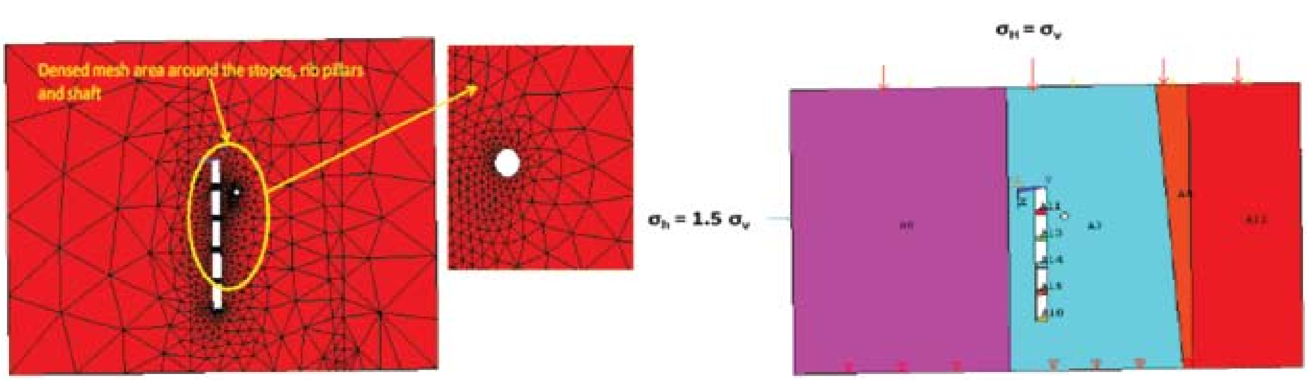
It can be observed that two corners of the model may experience failure due to shear (Figure 7). The intensity of plastic strain varies between 0 and 0.003165. Here, a higher value indicates severe failure of rock mass. This result signifies that concrete lining around the shaft is absolutely necessary. The following section deals with estimation of concrete strength of lining considering 0.3 m of lining thickness.

Strength of Lining Material

As mentioned before, biaxial far field stresses are applied in the finite element model. Due to this reason, non-uniform plastic zone has been developed around the shaft boundary as shown before. The critical radial presssure at various locations on the elasto-plastic boundary has been estimated as shown shcematically in Figure 8. From stresses obtained in Eluclidian plane, i.e., and are transformed into polar coordinates to calculate Pcr at different locations using the following stress transformation equations:

$$P_{cr} = \left(\frac{\sigma_{xx}+\sigma_{yy}}{2}\right)+\left(\frac{\sigma_{xx}-\sigma_{yy}}{2}\right)\cos 2\theta + \tau_{xy}\sin 2\theta \quad ...(7)$$

$$\sigma_{cc} = \frac{(2\times SF\times P_{cr})(a+t)^2}{(a+t)^2-a^2} \quad ...(8)$$



6a: Finite element mesh model

6b: Boundary condition is applied

Fig. : 6a and 6b

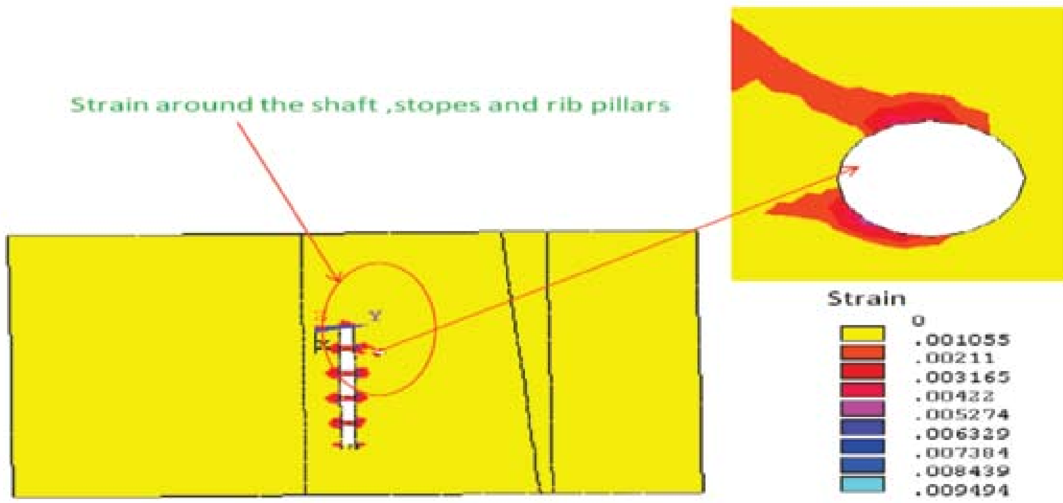


Fig. 7 : Distribution of intensity of plastic strain of horizontal section model

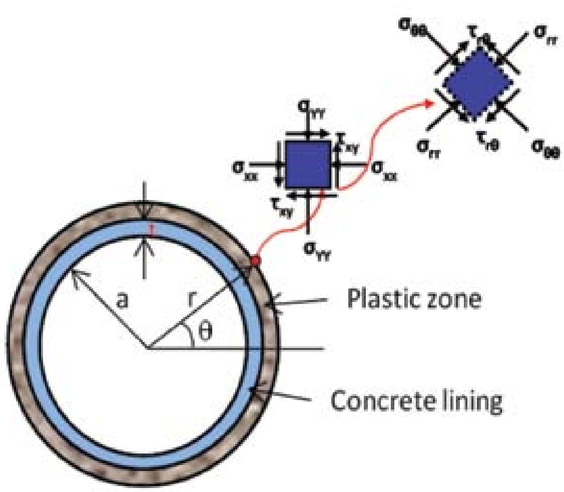


Fig. 8 : Schematic diagram of the critical radial pressure estimation.

where, σ_{cc} = Uniaxial compressive strength of concrete or shotcrete, SF = Factor of safety, t = Lining thickness, a = Finished radius of shaft.

The maximum value of the radial pressure is then taken as the critical pressure (Pcr) that may be considered for calculation of thickness or strength of lining material. Using the critical pressure value (Pcr), the required strength of the concrete lining of standard thickness of 0.3 m is calculated based on equation 9 (Brady and Brown, 2007). Table 4 lists the required compressive strength of the lining material for 0.3 m lining thickness having a safety factor of 1.5. It is found that compressive strength of concrete lining may vary from 16~48 MPa depending on the location. It is then receommended that on an average, the concrete strength for 0.3 m lining would be around 35~40 MPa. For this calculation, the elastic modulus of concrete is assumed to be 30 GPa.

Table 4 : Strengths of Lining of 0.3 m (SF = 1.5) in different locations

σ_{xx} MPa	-1.64	-2.3	-3.33	-4.32	-4.81	-5.63	-1.22	-1.46	-2.73	-1.13	-1.3
σ_{yy} MPa	-7.37	-7.98	-2.67	-7.05	-3.74	-2.82	-6.22	-8.94	-9.05	-7.83	-8.27
τ_{xy} MPa	0.13	-1.84	-2.51	-3.47	-3.5	-3.33	2.13	1.42	-0.51	1.58	1.44
P_{cr} MPa	1.71	1.81	0.94	2.65	1.71	0.98	1.16	1.62	2.79	2.2	2.68
σ_{cc} MPa	29.55	31.28	16.19	45.79	29.5	16.92	20.08	27.98	48.23	38.01	46.33

CONCLUSIONS

1. From 2D finite element analysis, it is found that failure zone may occur in the rock mass surrounding the unlined shaft, decline area and some part of the stopes. To minimize this effect, concrete lining is required around the shaft boundary. In this study, 0.3 m lining is required with a reinforced concrete having a compressive of at least 35~40 MPa.

2. The permanent lining of the decline is composed of reinforced concrete beam of 1m width and an I-beam of 20 mm made of steel is placed in between 2 concrete beams. Prior to erection of the permanent lining, the ground is stabilized by grouted fore-poling method. Ribbed steel rods of 38 mm diameter and 3 m in length are grouted longitudinally across into the roof with a spacing of about 200 mm. These grouted bolts are welded into the nearest I-beam. If required, a steel plate is also placed over the grouted bolts to arrest the fall of loose soil or rock material, if any.
3. It is recommended that a composite support consisting of a 20 mm thick steel beam and 0.5 m wide reinforced concrete beam placed on either side of the steel beam is considered to represent one set of permanent lining of the decline. This dimension and configuration as mentioned above is considered to be adequate for the current analysis, since the 1 m wide concrete lining coupled with a 0.2 m wide steel I-beam placed at half way in the width are repeated as permanent lining throughout entire length of the lined part of the decline.

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BIOGRAPHICAL DETAILS OF THE AUTHORS

Islavath Sreenivasa Rao graduated in Mining Engineering from the University College of Engineering, Kakatiya University, Telangana in 2009. He obtained M Tech in Mining Engineering from Indian Institute of Technology, Kharagpur, India in 2012. He worked as Mining Engineer in Bharat Aluminium Company Ltd (BALCO) from 2009 to 2010. In June, 2012, he joined M/s Singareni Collieries Company Ltd (SCCL), Telangana as Mining Graduate Trainee. From October, 2013, he has been Assistant Professor in Mining Engineering at the University College of Engineering, Kakatiya University, Telangana. His specialization in rock mechanics, coal mining methods and numerical modelling.

Debasis Deb is a Professor of the Department of Mining Engineering at Indian Institute of Technology (IIT) Kharagpur, India. He holds BTech (Hons.) in Mining Engineering from IIT Kharagpur. He received his MS and PhD degrees in Mineral Engineering and Interdisciplinary Mining and Engineering Mechanics from the University of Alabama, Tuscaloosa, USA.

Sujeet Bharti is currently a Research Scholar of the Department of Mining Engineering, IIT Kharagpur and obtained an M.Tech. degree from the same department. He is a recipient of MHRD scholarship during his postgraduate degree. He graduated in Mining Engineering discipline from Institution of Engineers (India) in the year 2011. He won the Institution Prize from Institution of Engineers (India) in graduation. He is an Associate Member of Institution of Engineers (India). He is also a Life Member of MGMI and MEAI. He has been given title of a Chartered Engineer (India) from Institution of Engineers (India). He has more ten years experiences in rock testing, analysis and numerical modelling in ISM Dhanbad and IIT Kharagpur.

INTERNATIONAL SOCIETY FOR ROCK MECHANICS

The **International Society for Rock Mechanics (ISRM)** was founded in Salzburg in 1962 as a result of the enlargement of the “Salzburger Kreis”. Its foundation is mainly owed to Prof. Leopold Müller who acted as President of the Society till September 1966. The ISRM is a non-profit scientific association supported by the fees of the members and grants that do not impair its free action. The Secretariat of ISRM is in Lisbon (Portugal). ISRM has 50 National Groups, including India.

The field of Rock Mechanics is taken to include all studies relative to the physical and mechanical behaviour of rocks and rock masses and the applications of this knowledge for the better understanding of geological processes and in the fields of Engineering.

The main objectives and purposes of the Society are to encourage international collaboration and exchange of ideas and information between Rock Mechanics practitioners; to encourage teaching, research, and advancement of knowledge in Rock Mechanics and to promote high standards of professional practice among rock engineers so that civil, mining and petroleum engineering works might be safer, more economic and less disruptive to the environment.

The President of ISRM is Prof. Xia-Ting Feng from China and the Secretary General is Dr. Luis Lamas from Portugal.

Important Positions held by Indians on ISRM Board

- Dr. T. Ramamurthy, Professor, IIT Delhi, served ISRM as its Vice President (Asia) during 1987-1991.
- Dr. Manoj Verman has been elected as ISRM Vice President at Large for the term 2011-2015
- Dr. V.B. Maji, Assistant Professor, Department of Civil Engineering, IIT Madras, has been selected for the ISRM Young Members Presidential Group.

ISRM Commissions

In 1967 the Council decided to appoint **Commissions** for studying scientific and technical matters of concern to the Society, operating in accordance with specific rules. In total 40 Commissions have been set for a limited period from then on. The ISRM Board has appointed sixteen Commissions for the period 2011-2015 in order to study some scientific and technical matters of topical interest to the Society.

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- Dr. Manoj Verman is President on ISRM Commission on “Hard Rock Excavation”
- Dr. K.G. Sharma, Professor, Department of Civil Engineering, IIT Delhi is representing on ISRM Commission on “Education”
- Mr. Asutosh Acharya, SO (F), Bhabha Atomic Research Centre, is representing on ISRM Commission on “Radioactive Waste Disposal”

ISRM Awards

Three awards have been instituted by ISRM, one in the memory of the founder of the Society (The Müller Award) the other in the memory of a recognized Past President. (The Rocha Medal) and the third is ISRM Lecture award

ISRM Fellows

The ISRM Council decided at its New Delhi meeting in October 2010, to create the status of Fellow, as the highest and most senior grade of membership of the ISRM. It is conferred on individuals, affiliated with the ISRM, who have achieved outstanding accomplishment in the field of rock mechanics and/or rock engineering and who have contributed to the professional community through the ISRM.

The appointment of ISRM Fellows is made by the ISRM Board.

The first group of Fellows was inducted in Beijing, during the 12^{th} International Congress held in October 2011 on Rock Mechanics.

INDIAN NATIONAL GROUP OF ISRM

Indian National Group of ISRM was got registered as the Committee of the International Society for Rock Mechanics (ISRM), under Societies Registration Act XXI of 1860 in the year 1991 and it represents the International Society for Rock Mechanics (ISRM) as Indian National Group, and has its Secretariat at the Central Board of Irrigation & Power (CBIP), Malcha Marg, Chanakyapuri, New Delhi.

The Indian national Group of ISRM, presently designated as ISRM (India), has been involved in dissemination of information on rock mechanics, mining and tunnel engineering by organising symposia, seminars, workshops, and training courses, both at national as well as international level, in liaison with international organizations, since its inception in 1991.

The Society aims to fulfill the objectives of ISRM. ISRM (India) is administered by the General Body and the Governing Council.

The office bearers of the Governing Council of Indian National Group of ISRM, for the term 2014-2017 are :

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ISRM (India) Journal

The Indian National Group of ISRM is publishing a Technical Journal **"ISRM (India) Journal",** on half yearly basis (January – June and July-December), since January 2012.

The aim of the journal is to encourage exchange of ideas and information among rock mechanics practitioners worldwide. The journal provides an information service to all concerned with Rock Mechanics about the development of techniques, new trends, experience gained by others to enable updating of knowledge. The original manuscripts that enhance the level of research and **c**ontribute new developments to the Rock Mechanics are encouraged. The journal is expected to exchange ideas and information between Rock Mechanics practitioners, help researchers, technologist and policy makers in the key sector of Water Resources, Infrastructure Development (including underground works), Hydro Power, Mining and Petroleum Engineering, etc. to enhance their understanding of it.

The Journal has both print and online versions.

Events Organised

1. Workshop on Rock Mechanics, May 1980, IIT Delhi
2. Indo Soviet Workshop on Rock Mechanics, July 1984, New Delhi
3. Workshop on Engineering Classification of Rocks, March 1985, New Delhi
4. Workshop on Rock Reinforcement, September 1986
5. Workshop on Rock Mechanics, 17-21 September 1990, New Delhi
6. Workshop on Rock Mechanics, 21-23 November 1991, New Delhi
7. Regional Workshop on Rock Mechanics, 18-21 May 1992, Guwahati
8. Regional Workshop on Rock Mechanics, 16-19 September 1992, Kochi

9. Regional Symposium on Rock Slopes, 7-11 December 1992, New Delhi
10. Training Course on Landslide Hazard Mitigation & Management, 5-10 April 1993, Guwahati
11. Regional Workshop on Rock Mechanics, 1-4 November 1993, Udaipur
12. Workshop on Blasting in Opencast Mining, 28 February 1994, Nagpur
13. Training Course on Landslide Hazard Mitigation & Management, 19-23 April 1994, Kozhikode
14. Workshop on Shotcreting, 23-24 November 1994, Vadodara
15. Workshop on Rock Mechanics, 22-25 March 1995, Panaji (Goa)
16. Workshop on Rock Mechanics, 12-14 August 1996, New Delhi
17. Workshop on Rock Mechanics & Tunnelling Techniques, 14-17 April 1999, Shimla
18. Workshop on Rock Mechanics & Tunnelling Techniques", September 2001, Kathmandu, Nepal
19. ISRM Regional Symposium – Advancing Rock Mechanics Frontiers to Meet the Challenges of 21 Century, 24-27 September 2002, New Delhi
20. Seminar on Productivity and Speed in Tunnelling, 26-27 June 2003, Dehradun
21. International Conference on Accelerated Construction of Hydropower Projects, 15-17 October 2003, Gedu, Bhutan
22. Conference on Development of Hydro Power Projects – A Prospective Challenge, 20-22 April 2005, Shimla
23. Workshop on Rock Mechanics & Tunnelling Techniques", 10-12 October 2007, Gangtok
24. Workshop on Rock Mechanics & Tunnelling Techniques", 24-26 April 2008, Manali
25. Workshop on "Applications of Rock Mechanics – Tools and Techniques", 15-17 January 2010, Nagpur (Maharashtra)
26. Seminar on "Rock Engineering", 8-9 March 2010, New Delhi
27. Seminar on "Meeting Rock Mechanics Challenge of Deep Underground Mining", 22-24 April 2010, Dhanbad (Jharkhand)
28. ISRM International Symposium 2010 and 6th Asian Rock Mechanics Symposium, 23-27 October 2010, New Delhi
29. Workshop on "Construction of Dams and Tunnels in Weak Rocks", 25-26 May 2011, JUIT, Waknaghat (Solan), Himachal Pradesh
30. Seminar "Grouting and Deep Mixing", 25-26 August 2011, New Delhi
31. Seminar on "Slope Stabilization Challenges in Infrastructure Projects", 20-21 October 2011, New Delhi
32. Seminar on "Geotechnical Challenges in Water Resources Projects", 19-20 January 2012, Dehradun (Uttarakhand)
33. International Seminar on "Survey and Investigations of Hydroelectric Projects – Issues and Challenges", 28th March 2012, New Delhi
34. Seminar on "Ground Control and Improvement", 20-21 September 2012, New Delhi
35. International Seminar on "Minimizing Geological Uncertainties and Their Effect on Hydroelectric Projects", 27-28 September 2012, New Delhi
36. Seminar on "Slope Stabilization Challenges in Infrastructure Projects", 29-30 November 2012, New Delhi
37. Seminar on "Geotechnical Challenges in Infrastructure Projects", 25-26 April 2013, New Delhi
38. Workshop on "Best Practices & Advancements in Geotechnical Investigations of Hydropower and Infrastructure Projects" – 24-25 July 2014, New Delhi
39. International Symposium on "Rock India 2014 – Present Technology and Future Challenges" and Workshop on "Open Pit Mining", 20-22 August 2014, New Delhi

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ISRM NEWS

THE ISRM ORANGE BOOK

“The ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 2007-2014”, known as the ISRM Orange Book, was published as a supplementary volume to the 2007 ISRM Blue Book. It is a product of the ISRM Commission on Testing Methods, chaired by Prof. Resat Ulusay, who is also the Editor of the book. The Orange Book was published by Springer and can be purchased by ISRM members at an advantageous price.

The Orange Book contains a total of 21 separate new and upgraded ISRM Suggested Methods that have been generated between 2007 and 2014. The Suggested Methods are collated in four parts, namely: “Laboratory Testing”, “Field Testing”, “Monitoring” and “Failure Criteria”. The Orange Book also includes two supplementary, but non-SM, documents. One of them is entitled “3D Laser Scanning Techniques for Application to Rock Mechanics and Rock Engineering”. The other supplementary document is titled ‘‘The Present and Future of Rock Testing: Highlighting the ISRM Suggested Methods’’.

VIII SOUTH AMERICAN CONGRESS ON ROCKS MECHANICS, 15-18 NOVEMBER 2015, BUENOS AIRES, ARGENTINA - AN ISRM REGIONAL SYMPOSIUM

SAIG, the Argentinean Society of Geotechnical Engineering will host the VIII South American Congress on Rock Mechanics (SCRM) in Buenos Aires (Argentina) from 15 to 18 November 2015.

The VIII SCRM is a Regional Symposium for South America of the ISRM. Previous congresses have been held in Colombia (1982, 2006), Brazil (1986, 1998), Venezuela (1990), Chile (1994) and Peru (2010). It is for the first time that this congress takes place in Argentina.

On this occasion, Buenos Aires-2015 will coincide with three important events to the geo-professionals. In parallel, organizes the 15th Pan American Conference on Soil Mechanics and Geotechnical Engineering (XV PCSMGE), the 6th International Symposium on Characteristics of Deformation of Soils (IS-BA2015) and the XXII Argentinean Congress of Geotechnical Engineering (CAMSIG XXII).

This synergy will bring together international experts, researchers, academics, professionals and geoengineering companies in a unique opportunity to exchange ideas and discuss current and future practices in the areas of soil mechanics, rock mechanics; and its applications in civil environmental engineering, and mining.

ROCK MECHANICS BASED ON AN ANISOTROPIC JOINTED ROCK MODEL

by Prof. Walter Wittke, ISRM Past-President

This book focuses on the fundamentals of rock mechanics as a basis for the safe and economical design and construction of tunnels, dam foundations and slopes in jointed and anisotropic rock. It is divided into four main parts:

- Fundamentals and models.
- Analysis and design methods.
- Exploration, testing and monitoring.
- Applications and case histories.

The anisotropic jointed rock model presented accounts for the influence of discontinuities on the stress-strain behaviour and the permeability of jointed rock masses. The applied analysis methods are based on the FE-method using computer programs developed by the author and his co-workers. Exploration, testing and monitoring are methods to evaluate the required rock mechanics parameters. Some case histories show the efficiency of the presented design philosophy.

Prof. Wittke is Emeritus of the Technical University of Aachen, Germany, and General Manager of the engineering company WBI Prof. Dr.-Ing. W. Wittke Beratende Ingenieure für Grundbau und Felsbau GmbH, which he founded in 1980. From 1979 to 1983, he was President of the International Society for Rock Mechanics (ISRM), and from 1990 to 2002 President of the German Society for Geotechnical Engineering (DGGT). During his professional life, he has been working as a designer and expert for numerous tunnels and caverns. Moreover, he has been dealing with dam foundations and slope stability. Along with these activities, he applied the rock mechanical models and the corresponding numerical analysis models, which he developed together with his co-workers. With the aid of back-analysis of the stresses and displacements measured during the construction and operation of the different structures, he continuously revised and improved these models.

WORKSHOP ON VOLCANIC ROCKS & SOILS, 24-25 SEPTEMBER 2015, ISLE OF ISCHIA, ITALY – *An ISRM Specialised Conference*

This ISRM specialised conference follows three previous Workshops on Volcanic Rocks, the last of which took place five years ago in Tenerife, Spain. The Workshop will provide a showcase for researchers to review recent developments and advancements in the geotechnical characterization and engineering applications related to volcanic formations. The conference topics are:

- Geotechnical characterization under both static and cyclic/dynamic loading conditions, with special regard to structural properties at different scales (technical classifications, microstructural features, laboratory characterization, field characterization, construction materials);
- Geotechnical aspects of natural hazards (slope stability, seismic risk, active volcanoes);
- Geotechnical problems of engineering structures (foundations, embankments, earthworks and dams, excavations and earth retaining structures, tunnels).

For detailed information, please visit the conference website at http://www.wvrs-ischia2015.it.

7th INTERNATIONAL SYMPOSIUM ON IN-SITU ROCK STRESS, TAMPERE, FINLAND, MAY 10-12, 2016 – *An ISRM Specialised Conference*

The Finnish National Group of ISRM and the Finnish Association of Civil Engineers RIL invite you to the 7th International Symposium on In-Situ Rock Stress, an ISRM specialised conference, to be held during 10-12 May 2016 in the beautiful city of Tampere, Finland. There have been six previous International Symposia on the rock stress topic, starting in 1976 in Sydney, Australia, and with the most recent one being held in Sendai, Japan, in 2013. 7th International Symposium on In-Situ Rock Stress, Tampere, Finland, May 10-12, 2016.

This symposium encompasses all aspects of rock stresses such as:

- Rock stress measurements with different methods
- Interpretation and analysis of results
- Case studies (nuclear waste disposal, mining, civil engineering)
- Regional stress fields
- Seismicity and rock stress
- New, innovated stress measurement methods
- Rock structures and rock stress
- Stress modeling

Abstract submission is done via internet. Abstract submission deadline is August 3 2015.

For more information, please visit the conference website at http://www.rs2016.org.

Workshop on BEST PRACTICES & ADVANCEMENTS IN GEOTECHNICAL INVESTIGATIONS OF HYDROPOWER AND INFRASTRUCTURE PROJECTS

24-25 JULY 2014, NEW DELHI

A Brief Report

Central Board of Irrigation and Power (CBIP), in association with Indian National Group of ISRM (ISRM-India) and Indian National Hydropower Association (INHA) organised a Workshop on "Best Practices & Advancements in Geotechnical Investigations of Hydropower and Infrastructure Projects", at CBIP Conference Hall, New Delhi during 24-25 July 2014.

In establishing hydropower projects, which involve large structures and underground works, there is demand for a well-considered program of investigation that would provide sufficient information, especially about the subsurface conditions towards establishing safe and economic project. Of the various activities, the geotechnical investigations are of paramount importance and being considered as a fundamental requirement of planning & design of civil engineering structures pertaining to hydroelectric projects and large infrastructure schemes, towards techno-economic feasibility evaluation. The extent of investigations depending on the stage of the project: a prudent approach being preliminary/ reconnaissance studies in the initial phase and detailing during subsequent phase/stage of investigation.

The objective of the Workshop was to provide a forum for executives, involved in development of hydropower and large infrastructure projects, to bring into focus the importance of planned geotechnical investigations, right from the initial Pre-feasibility stage, in duly selecting/finalizing project site alternatives, feasibility evolution and design formalization. The workshop focused to bring forth awareness about the best practices and advancements in instruments & techniques in geotechnical investigations, encompassing geological, geophysical and remote sensing studies as wells as geo-mechanical & other allied tests: It intends to foster the necessity of planned and extensive approach in geotechnical investigations, involving state-of-the-art techniques that would result in carrying out the project execution in challenging situations with enhanced level of confidence and facilitate in minimizing geological uncertainties towards establishing secure & economic project within schedule, with optimum involvement of direct explorations.

Following topics were discussed during the Workshop:

- Geological Investigation
- Geo-mechanical Testing
- Geophysical Survey
- Remote Sensing Study

More than 45 delegates participated in the Seminar.

The Workshop was inaugurated by Mr. R. Jeyaseelan, Former Chairman, Central Water Commission.

Dr. P.C. Nawani, President, Jindal Power Limited, Mr. S.K.G. Pandit, Chief Engineer, Central Water Commission, Mr. S.D. Dubey, Chief Engineer, Central Electricity Authority.

A Special Session on "Impact of Environment and Natural Disaster on Development of Hydro Power" was organized during the Workshop.

The eminent speakers, from the industry and research, included the following:

- Mr. Anish Mohan, Geologist, NHPC Ltd.
- Dr. P.K. Champati Ray, Indian Institute of Remote Sensing, Dehradun
- Dr. R.K. Gupta, Chief Engineer, CWC
- Mr. R. Jeyaseelan, Former Chairman, CWC
- Mr. S.L. Kapil, Chief (Geophysics), NHPC Ltd.
- Dr. P. C. Nawani, President, Jindal Power Limited
- Dr. Sanjay Rana, Director, Parsan Overseas Pvt. Ltd.
- Mr. Sanjiv Kumar, Chief (Geology), PHPA-II, Bhutan
- Mr. Imran Sayeed, Chief (Geology), NHPC Ltd.
- Mr. Ananda Sen, Former Chief (Geophysics), NHPC Ltd.
- Dr. V.M. Sharma, Director, AIMIL Limited

ROCK INDIA 2014

20-22 AUGUST 2014, NEW DELHI

A view of the dais

Dr. Yingxin Zhou, ISRM Vice President for Asia addressing the participants

Considerable activities in India in the field of rock mechanics are in progress, mainly due to the execution of projects for mining, irrigation, flood control, hydropower generation, road & rail tunnels in mountainous areas, sub-surface excavations for underground railway and storages. These activities often encounter the problems associated with unfavourable geological conditions. The experience gained during construction of these works will help in understanding the mechanism or rock support interaction, thus advancing the frontiers of rock mechanics.

Keeping the above in view, the Indian National Group of ISRM and CBIP, jointly organized an International Symposium “Rock India 2014 - Present Technology and Future Challenges” and Pre-Symposium Workshop on “Open Pit Mining” during 20-22 August 2014 in New Delhi.

The experiences during the events were shared by the professionals/scientists from Central Institute of Mining & Fuel Research, Engineers India Limited, Geoconsult India, Geological Survey of India, Hindustan Zinc Limited, Indian School of Mines, MOIL, National Institute of Rock Mechanics, The Singareni Collieries Company Ltd., etc.

Dr. Yingxin Zhou, ISRM Vice President for Asia, who was the Chief Guest, made presentation on the ISRM and Asian Rock Mechanics Council in the Opening Session of the Symposium on 21 August 2014.

RARE-2016
International Conference on
"RECENT ADVANCES IN ROCK ENGINEERING"

BENGALURU, INDIA, 16-18 NOVEMBER 2016

An ISRM Specialized Conference with Specialized Theme : Instrumentation & Field Measurements

The National Institute of Rock Mechanics will be organizing an international conference on rock mechanics, titled "Recent Advances in Rock Engineering", RARE-2016, as a Specialized Conference of the International Society for Rock Mechanics, during 16th to 18th November, 2016, at Bengaluru (earlier Bangalore), the beautiful and pleasant city in southern India.

The objective of the Conference is to bring together the practicing engineers, planning & design engineers, researchers, academicians and consultants in the field of rock mechanics and rock engineering. The Conference focuses on the developments and innovations made in recent years in the field of rock engineering, with special emphasis on field measurements, instrumentation and data collection, to understand the behavior of excavations in rock.

The Conference will address the problems and challenges posed in excavation of openings in rock for mining engineering, civil engineering & infrastructure development projects, and will discuss solutions to overcome these problems. Past experiences will be shared and application of new technologies will be explored.

The Theme & Sub-Themes of the Conference

The main theme of the Conference is recent advances in rock engineering; the specialized theme is instrumentation and field measurements. The specific topics will be:

DEVELOPMENTS IN TESTING & FIELD INVESTIGATIONS

- Rock mass characterization
- Geophysical probing techniques
- Stress regime around excavations
- Mechanical properties of rocks
- Instrumentation, and strata behavior monitoring

ADVANCES IN EXCAVATION & DESIGN METHODS

- Rock blasting & other excavation techniques
- Tunnelling methods
- Numerical modeling
- Rock reinforcement and ground improvement

CHALLENGES IN ROCK MECHANICS

- Deep mining
- Landslides & other natural hazards, and their mitigation
- Petroleum reservoir engineering
- Underground space utilization
- Other peripheral issues

Papers highlighting scientific and technological developments, those with innovative and futuristic ideas, and those reporting new developments and modern trends will be included in the Proceedings of the Conference. The deliberations will be more in the form of discussions. Webcasting of the proceedings will be made available for registered delegates who could not be present in Bengaluru.

For more details, please contact :

Dr. V. Venkateswarlu
Chairman, Organizing Committee, RARE-2016
& Director, National Institute of Rock Mechanics
PO : Champion Reefs – 563 117
Kolar Gold Fields, Karnataka, India
Ph : 91-8153-275000; Fax : 91-8153-275002
e-mail : dto@nirm.in; www.nirm.in

PUBLICATIONS OF INDIAN NATIONAL GROUP

PROCEEDINGS OF THE 6TH ASIAN ROCK MECHANICS SYMPOSIUM ON "ADVANCES IN ROCK ENGINEERING"

India is a fast developing economy requiring large scale infrastructure. Successive five year plans of Government of India have provided the policy frame work and funding for building up its wide infrastructure and manpower Rock Engineering does play an important role in most of infrastructure developmental activities related to civil engineering works. The need therefore is to keep ourselves abreast with the latest development in the field of rock engineering and its related fields such as design and construction of underground works, foundation of dams, slope stability etc.

Keeping this in view, ISRM International Symposium and the 6th Asian Rock Mechanics Symposium on **"Advances in Rock Engineering"** was jointly organized by the Indian National Group of ISRM and Central Board of Irrigation and Power, in New Delhi during 25-27 October 2010.

The proceedings of the Symposium contains extended abstracts of 166 papers from 27 countries selected for oral and poster presentations on the following topics:

- Testing and Modelling of Rocks & Rock Masses
- Slope Stability: Analysis & Design
- Foundations
- Underground Structures: Analysis, Design & Construction
- Artificial Intelligence
- Flow and Contaminant Transport
- Rock Dynamics
- Techniques for Improvement of Quality of Rock Mass
- Instrumentation and Monitoring

In addition, the proceedings contains the full texts of the following keynote lectures delivered by renowned experts from Australia, Canada, China, Israel, Italy, Japan, Singapore, U.K. and USA:

- On Site Visualization as a New Paradigm for Field Measurement in Rock Engineering
- Progress in the Understanding of Landslides from Massive Rock Slope Failure
- Deep Injection Disposal: Environmental and Petroleum Geomechanics
- Application of Intelligent Rock Mechanics Methodology to Rock Engineering
- Modelling Dynamic Deformation in Natural Rock Slopes and Underground Openings with Numerical DDA Method
- Underground Radioactive Waste Disposal — The Rock Mechanics Contribution
- Rock Dynamics Research in Singapore: Fundamentals and Practices
- The Large Open Pit Project
- Deep Underground Instrumentation and Monitoring

The full texts are available in electronic version with the extended abstracts in print.

MANUAL ON ROCK MECHANICS

The first Manual on Rock Mechanics, which was prepared under the guidance of an Expert Committee, was released by Central Board of Irrigation & Power (CBIP) in early 1979. The manual was very well received.

The manual was revised in 1988, to reflect the then state-of-art knowledge of Indian Engineers in the field of Rock Mechanics and contained 17 Chapters, covering basic concepts of Rock Mechanics, Field and Laboratory Tests on Rock Mass and Rock Specimen, Geophysical Investigations, Interpretation of Test Data and their Application to various problems of Foundation of Dams, Tunnelling, etc.

The Governing Council of the Indian National Group of ISRM felt that there was a need to update the manual, as more than 20 years had passed since its last publication.

Accordingly, the manual was updated and released during the ISRM International Symposium 2010 and 6th Asian Rock Mechanics Symposium held in New Delhi during 23-27 October 2010.

AIMS AND SCOPE

ISRM (India) Journal is a half yearly journal of the Indian National Group of International Society for Rock Mechanics (ISRM), which is involved in dissemination of information on rock mechanics, and its related activities in the field of foundation and abutments of dams, tunnel engineering, mining, underground works, rock slope stability, road works, etc.

The aim of the journal is to encourage exchange of ideas and information between rock mechanics practitioners worldwide. The journal provides an information service to all concerned with Rock Mechanics about the development of techniques, new trends, with a view to enable updating of knowledge. The original manuscripts that enhance the level of research and contribute new developments to the Rock Mechanics are encouraged. The journal is expected to provide useful information to researchers, technologist and policy makers in the key sector of Water Resources, Infrastructure Development (including underground works), Hydro Power, Mining and Petroleum Engineering, etc. The Journal has both print and online versions. Being peer-reviewed, the journal publishes original research reports, review papers and communications screened by the Editorial Board, consisting of renowned experts.

The manuscripts must be unpublished and should not have been submitted for publication elsewhere. **There are no Publication Charges**.

EDITORIAL BOARD

GUIDELINES FOR AUTHORS

This journal aims to provide a snapshot of the latest research and advances in the field of **Rock Mechanics.** The journal addresses what is new, significant and practicable. Journal of **ISRM (India)** is published twice a year (January-June and July-December) by Indian National Group of ISRM. The Journal has both print and online versions. Being peer-reviewed, the journal publishes original research reports, review papers and communications screened by Editorial Board.

The original manuscripts that enhance the level of research and contribute new developments to the Rock Mechanics are encouraged. The journal is expected to exchange the ideas and information between Rock Mechanics practitioners and help researchers, technologist and policy makers in the key sectors of **Water Resources, Infrastructure Development (including underground infrastructure), Hydro Power, Mining, and Petroleum Engineering, etc.,** to enhance their understanding of it. The manuscripts must be unpublished and should not have been submitted for publication elsewhere. There are no **Publication Charges.**

1. Guidelines for the preparation of manuscripts for publishing in "ISRM (India) Journal"

The authors should submit their manuscript in MS-Word (2003/2007) in single column, double line spacing as per the following guidelines. The manuscript should be organized to have Title page, Abstract, Introduction, Material & Methods, Results & Discussion, Conclusion, and Acknowledgement. The manuscript should not exceed 16 pages in double line spacing.

- Take margin as 1." (Left, Right, Top & Bottom) on A4 paper.
- The **Title** of the paper should be in bold and in Title case .
- The next item of the paper should be the author's name followed by the co-authors.
- Name of the corresponding author should be highlighted by putting an asterisk, with whom all the future correspondence shall be made.
- This should be followed by an affiliation and complete official addresses.
- Providing e-mail id is must.
- Please keep the title, author's name and affiliation center aligned.
- Use the following font sizes:

  Title: 14 point bold (Title Case), Author's name(s): 12-point bold, Author's Affiliations: 10-point normal, Headings: 11-point bold & caps, Sub-headings: 11-point normal & caps, Body Text: 10-point normal.
- The manuscript must be in English.
- Manuscripts are accepted on the basis that they may be edited for style and language.
- Use Times new roman as the font.
- Words used in a special context should appear between single quotation marks the first time they appear.
- Lines must be double-spaced (plus one additional line between paragraphs).
- Tables and figures must be included in the same file as the text in the end of the manuscript. Figures must be inserted into the document in JPEG or Tagged Image File Format (TIFF) format.
- Abbreviations should be spelt out in full for the first time they appear and their abbreviated form included in brackets immediately after.
- Communicating author will receive a soft copy of his/her published paper at free of cost.
- **Diagrams and Figures**: Only black & white figures are accepted. Figures should be entered in one column (center aligned) and should not exceed 6-inch total width. A minimum line width of 1 point is required at actual size. Annotations should be in Times New Roman 12 point with only the first letter capitalized. The figure caption should be preceded by 'Figure' followed by the figure number. For example, 'Figure 10.
- **Photographs and illustrations**: No color photographs are allowed. Image files should be optimized to the minimum possible size without compromising the quality. The figures should have a resolution of 300 dpi.
- **Equations** : Using the appropriate editor, each equation should appear on a new line. The equations referred to in the text, should be numbered sequentially with their identifier enclosed in parenthesis, right justified. The symbols, where referred to in the text, should be italicized.

$$E = mc^2 \quad ...(1)$$

- References: The papers in the reference list must be cited in the text in the order in which they appear in the text. In the text, the citation should appear in square brackets "[]". References of Journals, Books and Conferences must be written as shown in the example below.
  - Jones B., Brown, J., and Smith J. 2005, The title of the book. 1st edition, Publisher.
  - Jones B., Brown, J., and Smith J. 2005 The title of the conference paper. Proc Conference title 6: 9-17.
  - Jones B., Brown, J., and Smith J. .2005 The title of the journal paper. Journal Name. 3(4): 101-121.

Submission of Manuscript

The manuscript must be submitted in doc and pdf to the Editor as an email attachment to uday@cbip.org. The author(s) should send a signed declaration form mentioning that, the matter embodied in the manuscript is original and copyrighted material used during the preparation of the manuscript has been duly acknowledged. The declaration should also carry consent of all the authors for its submission to Journal of **ISRM (India)**. It is the responsibility of corresponding author to secure requisite permission from his or her employer that all papers submitted are understood to have received clearance(s) for publication. The authors shall also assign the copyright of the manuscript to the publisher **Indian National Group of ISRM.**

Peer Review Policy

Review System: Every article is processed by a masked peer review of double blind or by three referees and edited accordingly before publication. The criteria used for the acceptance of article are: contemporary relevance, updated literature, logical analysis, relevance to the global problem, sound methodology, contribution to knowledge and fairly good English. Selection of articles will be purely based on the experts' views and opinion. Authors will be communicated within Two months from the date of receipt of the manuscript. The editorial office will endeavor to assist where necessary with English language editing but authors are hereby requested to seek local editing assistance as far as possible before submission. Papers with immediate relevance would be considered for early publication. The possible expectations will be in the case of occasional invited papers and editorials, or where a partial or entire issue is devoted to a special theme under the guidance of a Guest Editor.

The Editor may be reached at: uday@cbip.org

ISRM SPONSORED FORTHCOMING EVENTS

- 14-16 October 2014, Sapporo, Japan. 8th Asian Rock Mechanics Symposium. The 2014 ISRM International Symposium.
- 4-5 November 2014, Sydney. The 3rd Australasian Ground Control in Mining Conference. An ISRM Specialised Conference.
- 8-10 November 2014, Xi'an, China. The 3rd ISRM International Young Scholars' Symposium on Rock Mechanics. An ISRM Specialized Conference.
- 10-13 May 2015, Montréal, Canada. ISRM 13th International Congress on Rock Mechanics.
- 6-8 September 2015, Wuhan, China. China Shale Gas 2015. An ISRM Specialized Conference.
- 24-25 September 2015, Island of Ischia, Italy. 4th Workshop on Volcanic Rocks and Soils. An ISRM Specialised Conference.
- 7-9 October 2015, Salzburg, Austria. EUROCK 2015 – Geomechanics Colloquy. An ISRM Regional Symposium.
- 15-17 November 2015, Buenos Aires, Argentina. XVIII South American Congress on Rock Mechanics. An ISRM Regional Symposium.
- May 2016, Cape Town, South Africa. African Rock Engineering Symposium. An ISRM Regional Symposium.
- 10-12 May 2016, Tampere, Finland. 7th In-Situ Rock Stress Symposium 2016. An ISRM Specialised Conference.
- 25-27 May 2016, Xi'an, China. GEOSAFE: 1st International Symposium on Reducing Risks in Site Investigation, Modelling and Construction for Rock Engineering - an ISRM Specialized Conference.
- 29-31 August 2016, Cappadocia. Turkey. EUROCK 2016. An ISRM Regional Symposium.
- October 2016, Bali, Indonesia. ARMS 2016 – 9th Asian Rock Mechanics Symposium. An ISRM Regional Symposium.

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- Pavement Strengthening and Rehabilitation
- Separation/Filtration/Drainage
- Lining Solutions

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Corporate Office
14th Floor, Vatika Professional Point,
Golf Course Ext. Road - Gurgaon – 122001
Tel - +91 124 4398400 I Fax - +91 124 4398444

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