ISRM INDIA

ISSN : 2277-131X (Print)
ISSN : 2277-1328 (Online)

ISRM (India) Journal

Vol. 1, No. 1 - January - June 2012

Half Yearly Technical Journal of Indian National Group of ISRM

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, experience gained by others 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 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. 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The manuscripts must be unpublished and should not have been submitted for publication elsewhere.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

INDIAN NATIONAL GROUP OF ISRM

ISRM (INDIA) JOURNAL

Volume 1, No. 1 *January 2012*

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Chanakyapuri, New Delhi – 110 021

MESSAGE

I feel greatly honoured to be elected as President of the International Society for Rock Mechanics (India) – ISRM (India) for the term till 2014. I thank all the Members of the Society for unanimously electing me as the President.

I thank my predecessor, Dr. K.G. Sharma, who guided the Society for almost 6 years, and was pioneer in bringing the Indian Chapter as one of the most active Chapter of the ISRM.

One of my first priorities, as President, is to increase its Membership, which I consider is necessary for overall growth of the Society.

On the Membership front, we stand 4th amongst all National Groups but there is need to further increase the Membership by bringing all concerned in its fold, and this would be possible only with the support of all concerned.

There is no reason why India cannot be amongst the best of the National Groups of the Society. I feel we need to bolster the Technical Activities of the Society such as Conferences; Workshops and Training Programmes ensuring high standards, besides providing all the necessary information to the participants. This would be possible by the cooperation and active support of all the Members.

I also take this opportunity to share with you that Dr. Manoj Verman has been elected as 3rd ISRM Vice President at Large for the term 2011-14. Dr. German has also been designated as President of ISRM Commission on “Hard Rock Excavation”. This is a moment of great pride for the Rock Mechanics Community of India.

The Inaugural Issue of ISRM (India) Journal is a step towards exchange of ideas and information between Rock Mechanics practitioners worldwide. I request all the readers to offer their valuable comments to enable us to improve its quality and utility.

I wish you all a Very Happy and Prosperous New Year – 2012. May the New Year bring you Excellent Health, Happiness and Best Success in your profession.

(Dr. H.R. Sharma)

President, ISRM (India)
&
Director (Design & Engg.), GVK Group

FROM THE EDITOR'S DESK

The International Society for Rock Mechanics (India), {ISRM (India)}, with its Secretariat at the Central Board of Irrigation & Power (CBIP), 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 is representing International Society for Rock Mechanics (ISRM). The ISRM has 48 countries as its National Groups, with India being one of them. On the membership front, Indian National Group stands 4th amongst all National Groups.

As part of the activities of Indian National Group, the first issue of its Technical Journal, ISRM (India) Journal, is in your hands. The Journal will be published on half yearly basis (January - June and July-December).

The aim of the journal is to encourage exchange of ideas and information between rock mechanics practitioners worldwide. The journal aim to provide 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 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.

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 to contribute technical papers/articles, news, etc., which would be of interest for publishing in the subsequent issues of the journal.

I also request for the comments/suggestions of the readers as to improve the utility of the Journal.

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

DYNAMIC PROPERTIES OF SANDSTONE ROCK SUBJECTED TO CYCLIC LOADING

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Abstract: *The paper deals with a brief geological and historical introduction of Ostrava-Karvina coalfield (OKC), which is followed by details about the rocks tested; macroscopic and microscopic petro-graphic characterization, their physical and geophysical properties obtained from laboratory seismic measurement. The rock specimens used to test their geophysical properties were representative of all kinds of rock samples from borehole Darkov 265-01. To find out possible relations with geophysical and physical properties, and to discover the fatigue properties of the rock, all these rock specimens (37 in total) were tested under the same-identical dynamic cyclic loading conditions of 1 Hz frequency and 0.1 mm amplitude using ramp waveform. The findings and various relations obtained from laboratory investigations are reported.*

1.0 INTRODUCTION

Dynamic loading in laboratory ultrasound measurements is typically accomplished by transmitting a stress wave-a sound wave or a shock wave, for example through the sample. Measurement of shear and compressional wave velocities is converted to moduli using standard procedures. Walsh (1993) has reported that for tested Westerly granite, static bulk modulus was about half the dynamic value at low pressure. The ratio of static to dynamic moduli is approached to unity with increase in confining pressure. According to Walsh, the presence of cracks in a sample under low pressure conditions is involved in the process causing the difference between the static and the dynamic moduli. This supposition is supported by the observation that the ratio of static to dynamic moduli was the lowest for rock types that had the largest crack densities.

In order to develop non-destructive methods in studying physical and mechanical properties of rock, various researchers (Soroush and Fahimifar, 2003; Starzec, 1999; Eissa and Kazi, 1988; Chong and Johnson, 1981, and others) have carried out different ultrasonic tests measuring velocity of P- and S-waves. Various relationships to estimate physical and mechanical parameters of rock by conducting simple and quick tests, like ultrasonic tests in the laboratory is provided by these researcher's. They have come up with various relationships to estimate physical and mechanical parameters of rock based on simple and quick tests.

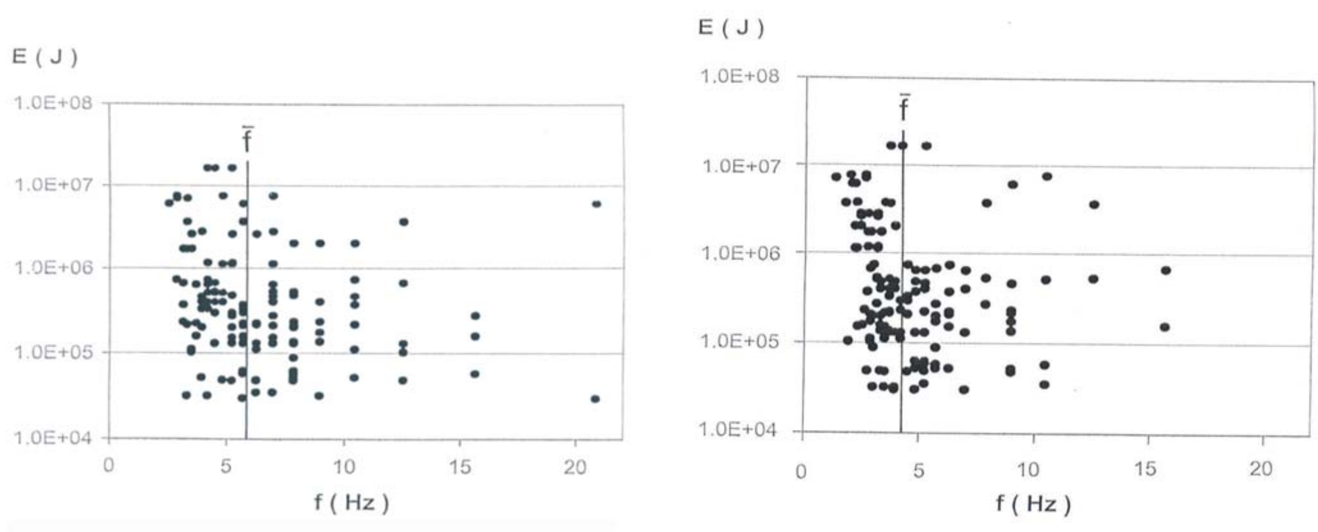
Many researchers in the past provided relationships by conducting laboratory tests in uni-axial compression mostly under static loading conditions, whereas dynamic properties were obtained from ultrasound tests in the laboratory. However, as far as the author's knowledge is concerned, none of these studies have centred around comparing dynamic properties with dynamic cyclic (fatigue) loading in uni-axial compression. In this paper, various relations are provided in-between dynamic properties obtained from ultrasound tests, physical properties and mechanical fatigue properties obtained by means of cyclic loading in laboratory investigations. The paper also deals with a brief geological and historical introduction of Ostrava-Karvina coalfield (OKC) that is followed by details about the rocks tested; macroscopic and microscopic petro-graphic characterization, their physical and geophysical properties obtained from laboratory seismic measurement. Rock specimens tested to obtain geophysical properties were representative of all kinds of rock samples obtained from rock-burst prone coal mine Darkov from OKC and from borehole Darkov 265-01. To find out possible relations with geophysical and physical properties, and to discover the fatigue properties of the rock, all these rock specimens (37 in total) were tested under the same-identical dynamic cyclic loading conditions of 1 Hz frequency and 0.1 mm amplitude using ramp waveform. The findings and various relations obtained from laboratory investigation are reported herewith.

2.0 GEOLOGY AND HISTORY OF OSTRAVA-KARVINA COALFIELD (OKC)

The Ostrava-Karvina coalfield (OKC) located in the Czech Republic is part of the Upper Silesian Coal Basin and is among the mining coalfields where, due to the given natural conditions, rock burst occurs. Hard coal seams have been mined in this area by underground mining methods for more than 200 years. The first rock burst was recorded in 1912.

Geological structure is quite variable. The deposit comprises of typical multi-seams and vertical distance between seams varies from a few meters up to several tens of meters. Rock burst started to be more and more frequent during the nineteen-seventies in part of the Karvina coalfield. The Karvina part is featured by a mean worked thickness of about 2.5 m (contrary to the present wind-up Ostrava section of the coalfield where it is about 1.1 m). The rock massif in the Karvina section, especially in saddle strata, consists mostly of solid siltstones, sandstones and conglomerates with uniaxial compressive strength of sandstone between 70 and 90 MPa. Compressive strength of coal is usually between 20 and 25 MPa, rarely even 35 MPa. Mining depth presently is about 800 m below the surface. During the last decade of the 20th century, drastic reduction in coal production occurred in OKC. Reduction of the annual coal production from about 24 million tonnes to about 14 million tonnes is manifested by the abandoning of mines in the Ostrava part. Within 25 years, i.e., between 1977 and 2002, no evidence is found of dependence between the volume of coal output (both total output and output of rock burst prone seams) and the frequency of rock burst. Thus, rock burst is obviously connected with geological and geomechanical conditions under which mining activity is carried out, and equally, with the intensity and success of continuously performed rock burst control measures. According to Konecny et al., (2003) rock burst in OKC is exclusively in-seam rock burst, directly linked with mine workings advanced in the same coal seam. A critical stress field is developed in the vicinity of the mine workings. A brittle failure of coal seam occurs at this event and the failure of roof strata and floor strata is minimal. When coal mining has passed into greater stratigraphic depths where above the coal seam N 530^0, a more than 100-160 m thick zone of rigid rocks without any continuously developed workable seams existed, the rock burst with its focus area outside the coal seam started to be manifested. The occurrence of critical stress field in such cases leads to brittle failure, not only of coal, but also with part of the rock massif. Such rock bursts are then, as a rule, a result of long-term and combined forming of unfavourable stress conditions in the given geological structure. Thus, rock bursts in OKC are considered as complex geomechanical events, in which the relationship between natural and technological factors and resultant manifestations is very complicated.

According to Petroš and Holub, (2003), the strongest rock burst that occurred during the last ten years in the OKC had P- and S- wave frequencies of about 5 Hz. Results of their detailed analysis and study (for more detail, see Petroš and Holub, 2003, Petroš et. al., 2004) are presented in Fig. 1 as a dependence of released seismic energy of rock burst versus prevailing frequency of P- and S-wave maximums. It is reported that frequencies of ground velocities of P- and S- waves are observed generally within the intervals of about 3-22 Hz and 2-16 Hz, respectively. However, according to them most occurred prevailing frequencies of P- and S- waves are observed mostly within the range of about 3-12 Hz and 2-9 Hz, respectively.



(a) P-waves (b) S-waves

Fig. 1 : Dependence of released seismic energy of rock burst vs. prevailing frequency of P- and S-waves for hypo-centric distance of 500-6000 m (after Petroš et al., 2004)

3.0 TESTED ROCK

The rock tested was mostly carboniferous sandstone obtained from the rock burst prone Darkov coal mine. The rock cores represent borehole Darkov 265-01. The samples were obtained from borehole cores in the saddle layer sandstone, at an approximate depth ranging from 930 to 980 m below surface in the case of Darkov. Samples were obtained from Darkov mine and sample No. 95-113 is referred to as Darkov 265-01-1 (then referred to as D1), from sample No. 114-118 as Darkov 265-01-2 (D2) etc., and hereafter, they are referred to as D1, D2, D3, D4, D5, D6, D7, D8 and D9, respectively. Details about various nomenclatures used to identify rock samples and their respective depth is provided in Table 1. The term "sample" refers to the block or cores of rock obtained from a site, while the term "rock specimen" refers to the specific piece of rock prepared and tested. Fig. 2 shows the prepared rock specimens from different samples obtained from borehole Darkov 265-01.

The tested rock specimens were of L/D ratio of 1 with an average diameter of 47.7 mm. Rock samples obtained from vertical borehole in the form of cores and rock specimens were prepared from the obtained cores and thus, loading direction in uniaxial compression testing in the laboratory was parallel to borehole axis. Rock specimens were prepared and tested according to ISRM testing procedure and guidelines (Fairhurst and Hudson, 1999).

Table 1 : Nomenclature used to identify rock samples with their respective depth and numbers of tested rock specimens

Sample	Identification	Symbol	Depth below surface (m)	Specimens
Borehole: Darkov 265-01, Mine: Darkov				
95-113	Darkov 265-01-1	D1	929-947	2
114-118	Darkov 265-01-2	D2	948-952	2
119-122	Darkov 265-01-3	D3	953-956	4
123-126	Darkov 265-01-4	D4	957-960	4
127-129	Darkov 265-01-5	D5	961-963	5
130-134	Darkov 265-01-6	D6	964-968	5
135-138	Darkov 265-01-7	D7	969-972	5
139-147	Darkov 265-01-8	D8	973-981	5
148-149	Darkov 265-01-9	D9	982-983	5

Fig. 2 : Prepared rock specimens showing sample disturbance from borehole Darkov 265-01

4.0 PETROGRAPHIC CHARACTERIZATION OF THE TESTED ROCK

The rock types are identified by the names of their respective locations. The colour, texture and mineralogical description are used to emphasise its predominant characteristics using macroscopic and microscopic petro-graphic analysis of the rock selected for testing. Detailed information is provided in the following sections.

4.1 Macroscopic Characterization

The macroscopic petro-graphic characterization of several randomly chosen rock specimens was carried out for all types of rock samples tested and is presented in Tables 2 to 4. After consulting these tables, it is found that these rock specimens were affected by coal intrusion, discontinuities and other geological disturbances. It is observed from the failed specimens in fatigue loading that traces and intrusion of coal fragments vary from specimen to specimen (Fig. 3). From Table 2, it is seen that tested rock specimens are comprised of a wide variety of minerals and textures, ranging from fine grained to coarse grained rocks, light gray to gray and whitish gray compact sandstone, as well as fine-gravel to medium-gravel grey conglomerate with matrix formed by sandstone etc. As it was found out, the tested rock specimens also had disturbance in terms of coal fragments, unoriented clast of coal, and other organic materials. In some cases, matrix was formed by conglomerate. This means that the tested rock specimens showed mineral content variation from specimen to specimen. This was also proved by X-Ray diffraction analysis, a result of which is provided in the following section.

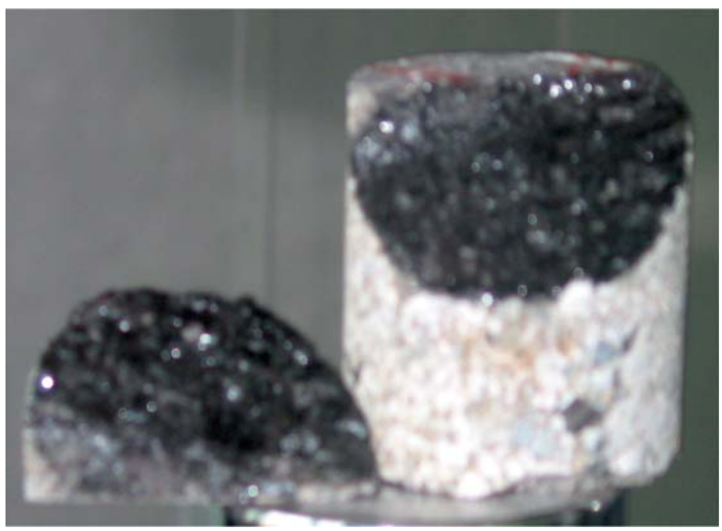


Fig. 3 : Sample disturbance in terms of coal intrusion in one of the tested rock specimen

Table 2 : Macroscopic petrographic characterization of rock samples and specimens from borehole Darkov 265-01

Sample/Specimen No.	Macroscopic petrographic characterization
D1/233	Medium-grained to coarse-grained, grey sandstone containing unoriented clasts of coal (up to 3 cm).
D1/310	Medium-grained, light grey sandstone with streaks (accumulated organic mass). The streaks are sporadically convergent, oriented diagonally (round 25°) to the axis of the core.
D2/15	Medium-grained, light grey, compact sandstone, with rare content of fragments (<3 mm) of dark material (siltstone). A diagonal fracture, probably filled with clay minerals, which goes through vertically.
D2/38	Fine-grained, white greyish, sandstone containing indistinct limited progressing streaks, perpendicular to the axis of the core. Sporadically there are partly small particles of coal.
D3/54	Medium-grained, grey, compact sandstone containing irregular clasts of coal.
D3/57	Medium-grained, grey, compact sandstone.
D4/33	Fine-grained to medium-grained, light grey, compact sandstone.
D5/33	Medium-grained, light grey, compact sandstone with increased content of feldspars. Some grains and fragments >2 mm.

D5/36	Fine-grained, light grey, compact sandstone with irregular small clasts of coal.
D6/26	Fine-grained, grey to dark grey, compact sandstone.
D6/80	Fine-grained, grey sandstone with smaller clasts of coal.
D7/9	Coarse-grained, light grey compact sandstone, it rarely contains pebbles >2 mm.
D7/15	Coarse-grained, light grey, compact sandstone.
D7/44	Fine-gravel, grey conglomerate with medium-rough pebbles. The pebbles are not well sorted out and the rounding is slight. The conglomerate is compact; matrix is formed by coarse-grained sandstone.
D7/52	Fine-gravel to medium-gravel grey conglomerate, pebbles were medium-rough, they were not sorted out. The conglomerate was polymict and matrix was formed by coarse-grained, sandstone. Compact.
D8/1	Fine-gravel, grey, polymict, compact conglomerate with predominance of quartz and quartzite. Matrix was formed by sandstone with abundance of feldspars.
D8/22	Fine-gravel conglomerate, matrix was formed by coarse-grained sandstone with abundance of feldspars, wrongly sorted out.
D8/28	Conglomerate with predominance of pebbles, matrix was probably formed by coarse-grained sandstone. The specimen contains clasts of coal, irregular in thickness from 1 mm to several millimetres. The conglomerate was polymict, with dominance of quartz pebbles.
D9/9	Coarse-grained, grey to light grey, compact sandstone, with expressive white feldspars. Some fragments > 3 mm.
D9/15	Medium-grained to coarse grained, wrongly sorted, compact sandstone with bold white feldspars. Sandstone contains schliers of organic substance.
D9/20	Contact of medium grained, light grey sandstone with coarse-grained. Grey, polymict sandstone and latter to conglomerate with matrix formed by coarse-grained sandstone.
D9/24	Coarse-grained light grey, compact, polymict sandstone, few fragments > 2 mm.
D9/43	Coarse-grained light grey sandstone with abundant of dark fragments and light grey feldspar.

4.2 Microscopic Characterization

Microscopic petrographic analysis using X-Ray diffraction analysis was carried out on several rock specimens representing various rock samples (Table 3). X-Ray diffraction analysis failed to identify coal content and it was identified as dickite bailey, a clay content material, and varied from 0.64 to 1.1%. The main mineral constituents in the case of Darkov sandstone rock samples were: ankerite from 0.26 to 2.3%, chlorite from 3.69 to 15.8%, muscovite from 22.1 to 40.8%, orthoclase from 6.85 to 18.42%, plagioclase from 2.96 to 5.24%, quartz from 19.59 to 60% and siderite from 1.44 to 8.27%. In the case of conglomerate rocks or sandstone with conglomerate matrix and vice versa, mineral content variation was found as: ankerite from 0.32 to 1.08%, chlorite from 4.14 to 8.35%, muscovite from 20.48 to 28.2%, orthoclase from 6.6 to 20%, plagioclase from 1.79 to 5%, quartz from 46.2 to 61.68% and siderite only 0.57% in one of the tested specimens. In the case of all types of sandstone rock samples, including conglomerate and sandstone with matrix formed by conglomerate, it is observed that the main constituent quartz, is varied from as low as 19.59% to as high as 61.68%. Based on the results presented above, it is possible to state that various rock samples and also rock specimens from the same sample had variation in mineral constituents. A detailed discussion about mineral content and its influence on rock properties is made while discussing the results.

Table 3 : Microscopic petrographic analysis using X-Ray diffraction analysis
(parentheses show standard error in percentage)

Sample/Specimen No.	Mineral content in %							
	Ankerite FeO.54	Chlorite IIb2	Dickite Bailey	Muscovite 1M	Orthoclase	Plagioclase Albite	Quartz	Siderite
D1/180 (Sandstone)	0.40 (0.42)	7.39 (2.25)	–	40.80 (5.70)	11.46 (1.80)	3.68(1.29)	34.80 (3.30)	1.44 (0.54)
D2/8 (Sandstone)	–	15.80 (4.50)	–	33.20 (7.20)	19.30 (3.60)	3.79(1.53)	19.59 (2.61)	8.27 (1.68)
D3/13 (Sandstone)	0.60 (0.66)	11.20 (3.60)	–	27.10 (5.70)	15.71 (2.88)	4.71(1.41)	36.00 (3.30)	4.65 (1.05)
D4/45 (Sandstone)	0.26 (0.51)	3.69 (1.14)	–	23.90 (4.80)	6.85 (1.17)	5.24(0.93)	60.10 (3.90)	–
D5/26 (Sandstone)	–	4.06 (1.71)	–	22.10 (4.50)	19.10 (3.00)	5.21(1.20)	49.50 (3.00)	–

D6/24 (Sandstone with coal intrusion)	–	7.43 (1.89)	1.03 (1.92)	27.70 (4.80)	18.42 (2.76)	4.66(1.14)	40.72 (2.91)	–
D6/81(Sandstone)	–	11.70 (3.60)	–	28.30 (4.50)	18.30 (3.00)	4.43(1.44)	31.88 (2.82)	5.34 (0.84)
D7/11(Sandstone with coal intrusion)	2.30 (0.75)	5.57 (1.56)	1.01 (1.26)	27.50 (4.20)	15.97 (2.49)	3.59(1.08)	44.08 (2.79)	–
D7/41(Conglomerate)	–	5.46 (1.71)	–	20.48 (2.40)	6.60 (1.05)	5.00(1.98)	61.68 (2.70)	0.57 (0.60)
D8/51(Sandstone-Conglomerate)	0.32 (0.33)	8.35 (2.82)	–	28.20 (4.50)	16.56 (2.67)	3.68(1.17)	42.90 (3.00)	–
D8/123 (Conglomerate with coal intrusion)	1.08 (0.63)	5.66 (1.77)	0.64 (0.63)	20.40 (4.20)	19.47 (2.67)	3.48(1.20)	49.30 (3.30)	–
D9/22 (Sandstone)	–	5.75 (1.26)	–	27.70 (3.90)	20.26 (2.73)	2.96(1.47)	41.24 (2.91)	2.13 (0.72)
D9/45 (Sandstone-Conglomerate)	0.48 (0.33)	4.14 (1.53)	–	27.40 (4.80)	20.00 (3.30)	1.79(1.08)	46.20 (3.30)	–

5.0 PHYSICAL PROPERTIES

Physical rock properties determined included dry density, saturated density, specific gravity, mass moisture content, porosity and water absorption according to the method suggested by ISRM (1981). The average physical properties of different rock samples are provided in Table 4. Density and specific gravity is the intrinsic physical property that denotes the heaviness of the mineral content of the rock in its unit volume. This is influenced by the type of mineral, discontinuities and type of fluid saturation. Also, water absorption depends on the mineralogical qualities and porosity of the rock in question. Rock is composed of one or more minerals in granular form. All these grains are in intact position due to grain interlocking and cementing materials consisting of other minerals, cohesive granular aggregates, and moisture present therein. The structure of a grain network, unlike that of the crystal lattice in a single grain, is rarely homogeneous and periodic. The strength of rock depends on the strength of the constituting minerals, as well as the cementing materials. At the microscopic level, the strength of rock depends on the presence of cracks, voids and inter-granular features.

From Table 4, it is observed that various physical properties and standard deviations mentioned vary considerably. Porosity is found to vary from 5.9 to 10.1%. The observed variation in values of various physical properties is due to variation in mineralogical composition, porosity and intra-structure among the different rock samples and even in different specimens of the same rock samples. From this, it can be concluded that rock specimens from the same sample type also had variation in mineral constituents and cementing material, sample disturbance in terms of microcracks, voids, pores and inter-granular features, coal intrusion and other organic materials. It is well known that mechanical properties of rock samples measured under laboratory conditions depends on the strength of the constituting minerals, as well as cementing materials and sample disturbance. The possible effect of all these on fatigue rock properties is presented while discussing results.

6.0 GEOPHYSICAL PROPERTIES FROM LABORATORY SEISMIC VELOCITY MEASUREMENT

Geophysical measurement, particularly with the use of seismic pulse techniques, is more and more universally used to determine elastic properties of rock and rock masses under laboratory conditions as well as in situ. Seismic methods depend primarily upon the

Table 4. Average physical properties of the rocks *(parentheses show standard deviation)*

Sample No.	ρ_d(Dry density) (kg/m³)	ρ_s(Saturated density) (kg/m³)	G (Specific gravity) (kg/m³)	w (Mass moisture content) (%)	η (Porosity) (%)	S (Water absorption)(%)
D1	2492 (39)	2582 (21)	2746 (55)	0.29 (0.02)	9.2 (1.6)	2.1 (0.16)
D2	2534 (47)	2591 (16)	2748 (24)	0.49 (0.12)	7.7 (2.9)	2.0 (0.20)
D3	2568 (49)	2571 (52)	2762 (22)	0.45 (0.12)	5.9 (1.2)	1.7 (0.14)
D4	2470 (27)	2510 (25)	2730 (46)	0.22 (0.03)	10.1 (2.5)	2.3 (0.16)
D5	2459 (26)	2526 (21)	2700 (10)	0.25 (0.05)	9.0 (1.0)	2.3 (0.26)
D6	2511 (20)	2553 (24)	2760 (23)	0.70 (0.09)	9.3 (1.2)	2.0 (0.10)
D7	2500 (24)	2537 (20)	2750 (21)	0.46 (0.05)	9.4 (2.5)	1.8 (0.24)
D8	2484 (31)	2522 (29)	2710 (29)	0.51 (0.07)	8.6 (2.9)	1.8 (0.25)
D9	2525 (33)	2553 (29)	2736 (20)	0.65 (0.05)	8.5 (2.5)	1.8 (0.38)

measurement of the velocity of propagation of ultra-sound waves through rocks and associated materials. For most applications the following types of body waves are of particular interest (McDowell, 1993).

(i) Compressional or longitudinal waves (P-waves). These have a particle motion in the direction of wave propagation and travel faster than all other waveforms.

(ii) Shear or transverse waves (S-waves). Ground motion is transverse to the direction of wave motion, i.e., polarized in one plane. In practice, the S-wave motion is normally polarized into components which are parallel and perpendicular to the surface of the ground, i.e. S_H - and S_V-waves.

The velocity of seismic waves depends upon a large number of factors, the most important being the elastic moduli. For homogeneous isotropic elastic media, P- and S-wave velocities are related to dynamic elastic moduli and bulk density, using the following formulae:

$$E_d = \frac{\rho.V_p^2.(1+\mu_d).(1-2\mu_d)}{(1-\mu_d)} \quad ...(1)$$

$$\mu_d = \frac{(V_p^2 - 2V_s^2)}{2(V_p^2 - V_s^2)} \quad ...(2)$$

$$E_d = 2.\rho.(1+\mu_d).V_s^2, \quad ...(3)$$

where,

V_p and V_s are P- and S-wave velocities (m/s),

E_d is Young's dynamic modulus of elasticity (N/m^2),

ρ is bulk density (kg/m^3), and

μ_d Poisson's dynamic ratio.

Although most soil and rock masses are not homogeneous and isotropic, they are usually considered to behave elastically where stress levels are low, and wavelength is significantly greater than the dimensions of pore spaces, voids and fractures. Therefore, when deriving dynamic elastic parameters from seismic velocity values, it is very important to consider the relationship between wavelength and the size, spacing, infill, etc., of fractures. Although these formulae suggest that velocity will decrease with increasing density, the opposite is normally the case, because an increase in density is usually related to an increase in compaction and or cementation, resulting in an even more significant increase in elastic moduli (McDowell, 1993).

The average elastic dynamic properties of the rock tested in the laboratory, using seismic velocity measurements, are provided in Table 5. Altogether 37 rock specimens were tested, representing all kinds of samples from borehole Darkov 265-01. The wide variation in values of elastic dynamic properties is due to variations in mineralogical composition, porosity and intra-structure among different rock specimens and even in different samples coming from the same borehole. From Table 6 it can be seen that changes in values from dry to saturated condition differ greatly, the percentage being as much as 20% in the case of P-wave velocity and 29% in the case of dynamic elastic modulus. According to Martin and Stimpson (1994), P-wave velocity is sensitive to crack density, and hence, is a good indicator of sample disturbance. The higher the sonic velocity is, the stiffer the rock. The difference in the values of sonic wave velocity for dry and saturated conditions shows the presence of micro-fissuring in the tested rock specimens. From this, it can be further concluded that the rock specimens tested showed disturbances in terms of mineralogy, coal intrusion, joints, cracks, etc.

Table 5 : Dynamic properties obtained by seismic velocity measurement *(parentheses show standard deviation)*

Sample (Specimen tested)	Dry rocks				Saturated rocks			
	V_{pd} (m/s)	V_{sd} (m/s)	μ_{dd}	ρ_{dd} (GPa)	V_{ps} (m/s)	V_{ss} (m/s)	m_{ds}	E_{ds} (GPa)
D1 (2)	3256 (109)	2268 (38)	0.03 (0.03)	27 (2)	3875 (135)	2222 (40)	0.17 (0.01)	33 (1)
D2 (2)	3438 (34)	2368 (30)	0.05 (0.01)	30 (1)	4138(50)	2316 (59)	0.22 (0.01)	39 (1)
D3 (4)	4056 (233)	2638 (111)	0.13 (0.03)	40 (3)	4384 (322)	2564 (161)	0.19 (0.01)	45 (5)
D4 (4)	3638 (145)	2396 (87)	0.11 (0.04)	32 (3)	4235(73)	2450 (33)	0.20 (0.01)	41 (2)
D5 (5)	3679 (173)	2465 (92)	0.10 (0.03)	33 (3)	4285 (173)	2473 (94)	0.20 (0.01)	42 (4)
D6 (5)	3666 (53)	2422 (60)	0.11 (0.03)	33 (1)	4187 (113)	2430 (94)	0.20 (0.04)	40 (2)
D7 (5)	4439 (180)	2864 (167)	0.14 (0.06)	47 (4)	4775 (207)	2752 (109)	0.20 (0.01)	52 (4)
D8 (5)	3740 (654)	2673 (120)	0.15 (0.03)	41 (5)	4304 (609)	2666 (156)	0.21 (0.01)	49 (6)
D9 (5)	3965 (269)	2588 (98)	0.13 (0.04)	38 (4)	4396 (263)	2527 (126)	0.20 (0.02)	44 (6)

Table 6 : Percentage change for properties given in Table 5

Sample	Change in values from dry to saturation (%)			
	V_p	V_s	μ_d	E_d
D1	19	-3	262	17
D2	20	-2	360	29
D3	8	-3	49	12
D4	17	0.03	83	25
D5	17	-0.34	107	25
D6	14	0.31	80	23
D7	8	-4	125	11
D8	16	-0.3	45	21
D9	11	-3	28	7

6.1 Relationship between P-and S-Wave Velocities

Firstly, the data related to P- and S-wave velocities that were obtained from measurements on 37 rock specimens, representing all kinds of rock samples from borehole Darkov 265-01, in dry and saturated conditions, are analysed. Relationship between V_p and V_s is shown in Figs. 4 and 5. From these figures, the linear relationship between P- and S-wave velocities is clearly visible. Because of a high regression coefficient between these parameters, it is easily possible to estimate one velocity when knowing the other.

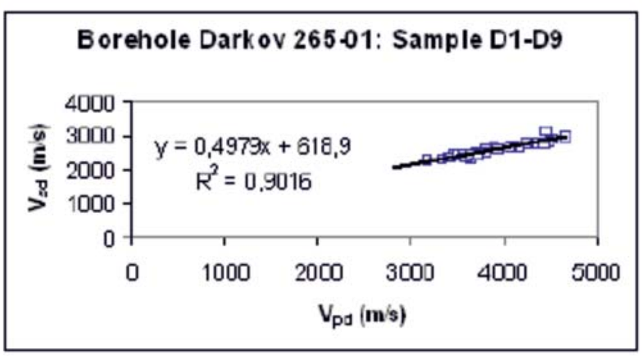


Fig. 4 : Relationship between dry P-and S-wave velocities

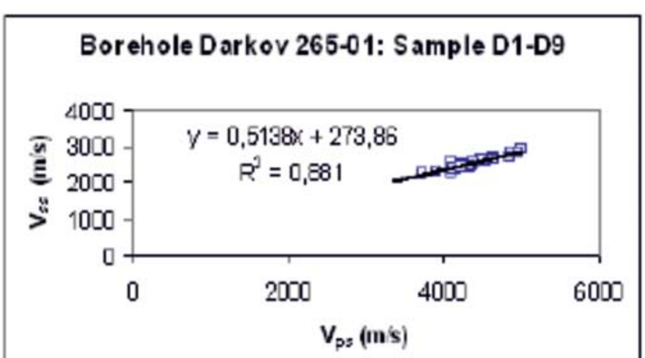


Fig. 5 : Relationship between saturated P-and S-wave velocities

6.2 Relationship between Geophysical and Physical Properties

The relationship between different P- and S-wave velocities and with that of physical properties is tried. The relationship between dry density and dry P-wave velocity is shown in Fig. 6. An increase in P-wave velocity with increasing density is observed. This is obvious since wave velocities generally increase with increasing specimen density. Fig. 7 shows the relationship between water absorption and dry P-wave velocity. It shows an increase in dry P-wave velocity with decrease in water absorption. Water absorption depends on mineralogy and porosity of rock. The ultra-sound velocity depends on the same factors. Therefore, it would be possible to find a relationship between water absorption and velocity. The scatter in the data and poor correlation could be associated with sample extraction, disturbance, sample preparation and measurements.

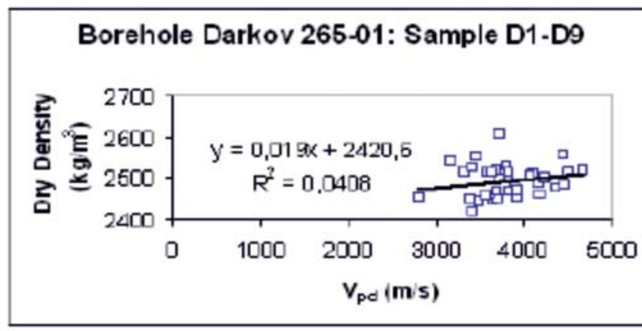


Fig. 6 : Relationship between dry density and dry P-wave velocity

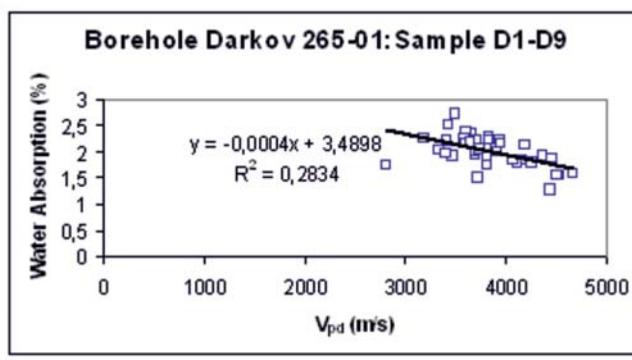


Fig. 7 : Relationship between water absorption and dry P-wave velocity

7.0 RELATIONSHIP BETWEEN FATIGUE AND PHYSICAL AND GEOPHYSICAL PROPERTIES

To find out possible relations with geophysical and physical properties and with fatigue properties of the rock, all these specimens (37 in total) were tested under the same dynamic cyclic loading conditions. Simulated loading frequency was 1 Hz, and amplitude was 0.1 mm, using ramp waveform. For more detail about the testing equipment and loading condition kindly see author's paper (Bagde and Petros, 2009, 2005a,b,c). During these tests, the laboratory was equipped with a room temperature control system and thus maintained a temperature of 20°C. Rock specimens were in water saturation to conduct seismic velocity measurements on saturated rock specimens. Uniaxial compression tests were carried out on these specimens in dry conditions. The results of these tests are provided in Table 7. Before conducting the tests, specimens were first oven dried for 24 hrs at a temperature of 60°C and then cooled in desiccators to obtain constant weight before testing. Findings from these laboratory investigations are also presented in the following.

Table 5 : Fatigue properties from uni-axial compression tests in cyclic loading (***parentheses show standard deviation)***

Sample (Specimen tested)	σ_{fp} **(MPa)**	E_{avd} **(GPa)**
D1 (2)	124.98(8.08)	9.61(0.64)
D2 (2)	135.85 (8.41)	10.87(0.44)
D3 (4)	130.45 (5.47)	11.61(0.58)
D4 (4)	115.56 (17.57)	9.67(1.22)
D5 (5)	118.08 (19.51)	10.34(0.86)
D6 (5)	127 (4.57)	10.83(0.36)
D7 (5)	106.26 (7.43)	11.34(0.53)
D8 (5)	117.85 (23.5)	11.24(1.67)
D9 (5)	117.92 (20.86)	10.84(1.32)

7.1 Relationship between Fatigue and Physical Properties of Rock

The relationship between peak fatigue strength ($_{fp}$) and dry density shows an increasing trend (Fig. 8). It means that the denser the rock is, the higher the strength. Density of a rock is an intrinsic physical property that denotes the heaviness of the mineral content of the rock in its unit volume. This is influenced by the type of minerals, discontinuities and the type of fluid saturation. The relationship between peak fatigue strength and water absorption (Fig. 9) shows a decreasing trend, which means that the more water absorption is more porous and less denser is the rock, thus the lower the strength.

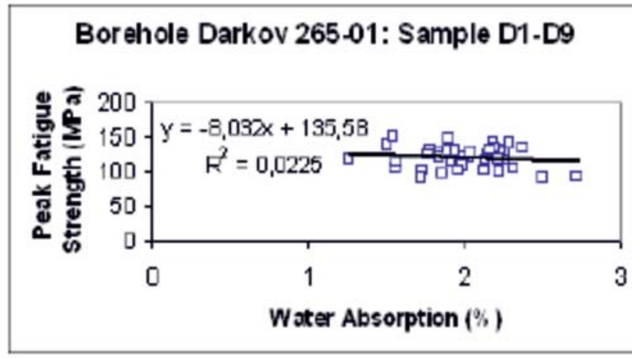


Fig. 9 : Relationship between peak fatigue strength and water absorption

Average Young's modulus shows an increasing trend with dry density (Fig. 10), while it shows a decreasing trend with water absorption (Fig. 11). Water absorption is a good indicator of mineralogical qualities and porosity. The modulus is a good indicator of rock type and depends on the mineralogical qualities or constituents of the material. From this it can be concluded that tested rock specimens from various samples showed wide variation in mineralogical qualities and sample disturbance, and thus failed to produce an adequate relation coefficient. For this reason, it can be concluded that fatigue properties of the rock under dynamic cyclic loading conditions are very sensitive to mineralogical qualities and texture of the rock type. In literature, it is often described that engineering behaviour of rocks is found to depend on their intrinsic properties, such as mineralogical composition, cementing material, grain size and shape, texture and porosity. Pore structure of rock also significantly influences its behaviour in terms of its strength and durability, permeability and mechanical properties of porous materials are closely related to their pore structure (Bagde, 2000).

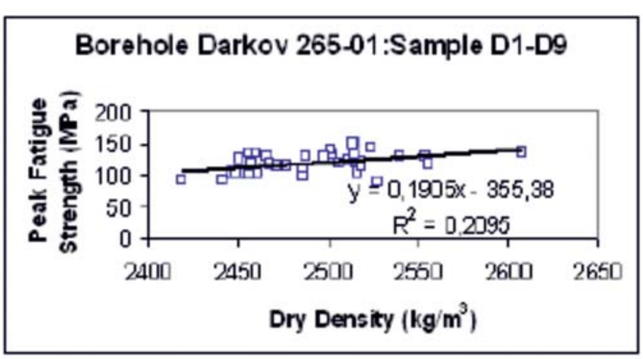


Fig. 8 : Relationship between peak fatigue strength and dry density

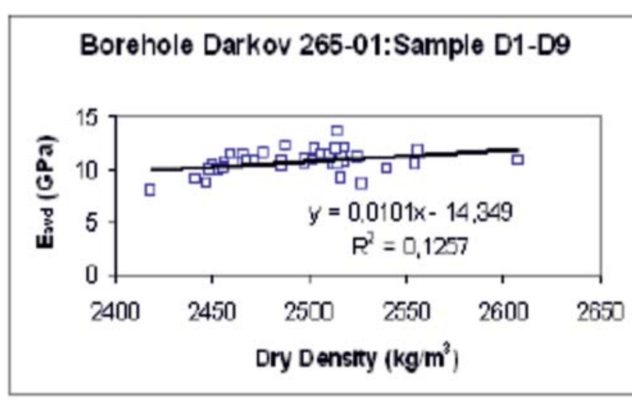


Fig. 10 : Relationship between Average Young's modulus and dry density

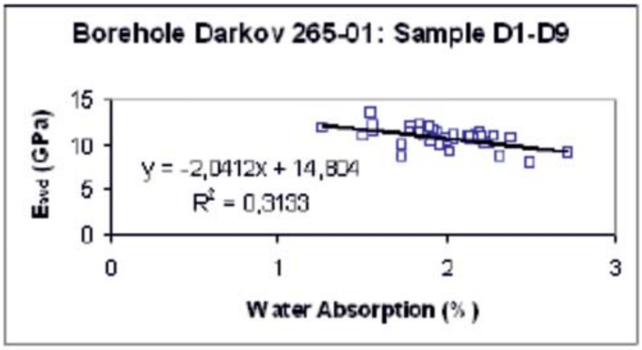


Fig. 11 : Relationship between Average Young's modulus and water absorption

7.2 Relationship between Fatigue and Geophysical Properties of Rock

The relationship between peak fatigue strength and that with dry P- and S-wave velocity is shown in Figs. 12 and 13, respectively. It is seen that hardly any relation exists between these parameters, since the obtained relation coefficient is very low. The relationship between average Young's modulus and that with dry P- and S-wave velocity is shown in Figs. 14 and 15, respectively. It shows an increase in average Young's modulus with an increase in P- or S-wave velocity. Also, the obtained relation coefficient is at 0.40 and 0.30 in the case of P- and S-wave, respectively, with average Young's modulus. Fig. 16 shows the relationship between average Young's modulus from dynamic cyclic uniaxial compression loading under laboratory conditions, and dynamic modulus obtained from seismic velocity measurement. The plot shows an increasing trend where obtained relation coefficient is 0.40. The obtained relation coefficient in all the above cases is not good statistically, but it shows some trend with different properties. One of the reasons of data dispersion in this case is the effect of rock type or mineralogy variation. As mentioned earlier, the tested rock specimens varied greatly in terms of mineralogy, texture, structure and sample disturbance, and thus it is hypothesized that dynamic cyclic loading conditions are very susceptible to rock type in terms of mineralogy, texture and structure.

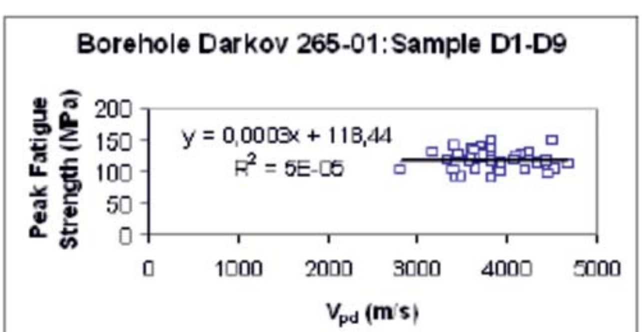


Fig. 12 : Relationship between peak fatigue strength and dry P-wave velocity

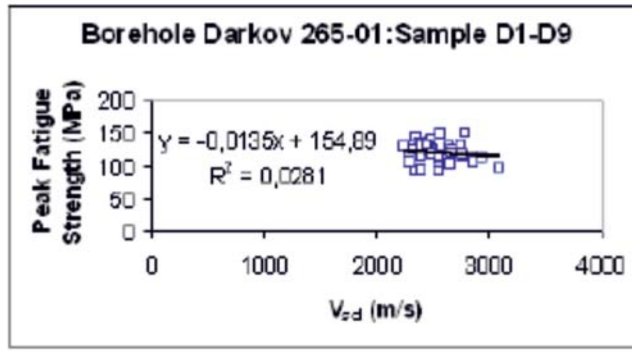


Fig. 13 : Relationship between peak fatigue strength and dry S-wave velocity

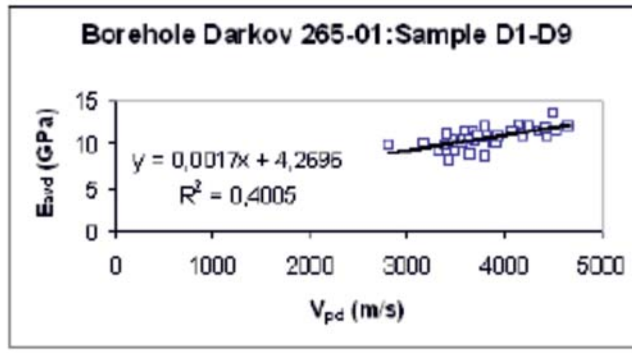


Fig. 14 : Relationship between average Young's modulus and dry P-wave velocity

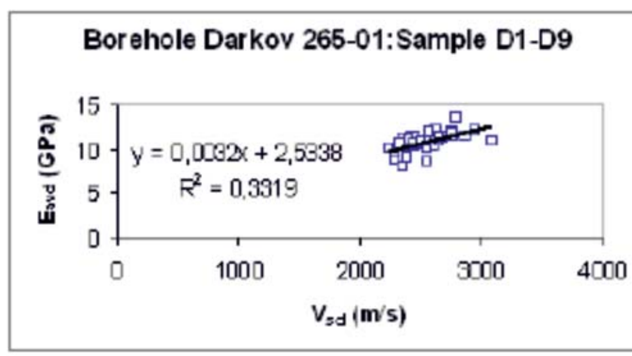


Fig. 15 : Relationship between average Young's modulus and dry S-wave velocity

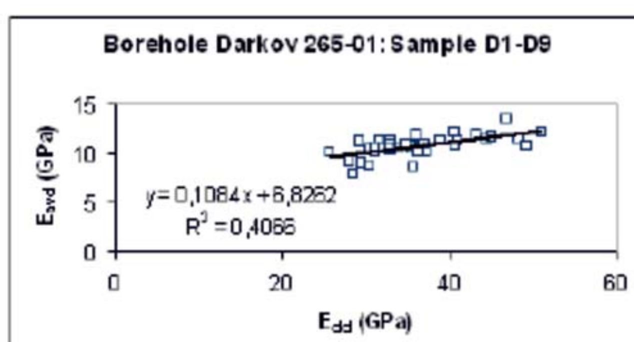


Fig. 16 : Relationship between average Young's modulus and dynamic modulus from seismic velocity measurement

Rock mass can have a high degree of variability in rock types, properties and can contain extensive geological features, such as dykes and faults, or sets of discontinuities, such as bedding planes, joints, etc. Dimensions of rock mass structure are very large compared to the maximum possible specimen dimensions used under laboratory conditions. Therefore, due to inherent material variability in rock mass, laboratory results obtained from one specimen to another are likely to vary widely, even though the specimens might have been collected on nearby locations in a structure. Giving the example of the pegmatite and granodiorite rock samples, Eberhardt (1998) pointed out that though the pegmatite was found to have the lowest density of the three rock types tested; it had similar velocities to those of the granodiorite specimens. The fact that they were larger than expected velocities could be attributed to the much larger crystals found in the pegmatite and larger crystals mean fewer grain boundaries, which act to reduce the velocity of the acoustic pulse. Thus Eberhardt noted that in such extreme cases, physical properties of the individual minerals might control, or partly control, the overall behaviour of the sample.

The process of drilling and recovering core for laboratory testing often results in sample disturbance through stress-induced micro-fracturing, altering the physical properties of the rock. This disturbance may be the result of mechanical abrasion and vibration due to the drilling process itself, and/or through stress relief cracking in cases where the samples are retrieved from high in situ stress regimes. In general, the extent of this disturbance is often a function of drilling depth and, to a lesser degree, borehole orientation. For example, in situ stresses regimes generally increase with depth resulting in higher crack densities in the retrieved samples. Martin and Stimpson (1994) noted that it then becomes possible for samples of the same rock type, but obtained from different in situ regimes, to have drastically different mechanical properties.

8. CONCLUSION

The Ostrava-Karvina coalfield (OKC) is classified as rock burst prone and having complex geo-mechanical structure. The sandstone rock samples studied were from Darkov coal mine from the OKC, and they represent the rock massif of a rock burst prone coal mine in the Czech Republic. From the petro-graphic characterization, physical and geophysical properties, samples and rock specimens are considered to embody a high degree of sampling disturbance in terms of mineralogy, texture and structure. This is also confirmed by various presented relationships between physical, geophysical and fatigue rock properties obtained through laboratory testing. Finally, it can be concluded that dynamic cyclic loading conditions are very susceptible to rock type in terms of mineralogy, texture and structure.

ACKNOWLEDGMENTS

This work has been performed with financial grant No. 105/01/0042 from Grant Agency of the Czech Republic. The study presented here is part of the Ph. D. thesis submitted to Faculty of Mining and Geology, VSB-Technical University, Ostrava, Czech Republic, by the first author in the year 2004.

The first author would like to take this opportunity to thank the Czech Government for providing a scholarship to pursue his doctoral study in the Czech Republic. Special appreciation is also extended to Doc. Ing. L. Hofrichterova from the Department of Geophysics for her help in carrying out sonic tests, Dr. A. Kozusnikova from the Institute of Geonics, for her help in carrying out macroscopic petrographic analysis, and Dr. Ing. Dalibar Matysek for his help in carrying out X-Ray diffraction analysis. Thanks are due to Doc. Ing. P. Koneèný from the Institute of Geonics, ASCR, Ostrava for his valuable criticism and suggestions. Thanks also belong to the library staff who were of great help. Thanks also to my friend Ing. Miloš Daniel for his programming help and to Ing. P. Michalèík and Ing. O. Špinka for their help in carrying out experiments and Mrs. Hanka Kolarova for her editing help. I also wish to thank my wife Shaila and son Anurag for living without me and taking pain in their stride and for their encouragement throughout this study.

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TUNNELS IN WEAK GROUND : DISCRETE ELEMENT SIMULATIONS

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ABSTRACT *: Tunnels are an absolute necessity in the modern world where the underground space utilization is significant for the development of a nation. The construction of tunnels in an already congested area is very risky and requires special skills. If tunnels are built near existing structures or on shallow soft ground, there is possibility of very large deformations of the ground. This may affect the safety of the existing structures as well as the tunnels. Also the stresses developed during the construction of the tunnels also play a significant role in the tunnel stability. This stresses the fact that proper supports and reinforcements are necessary to ensure the safety of the tunnel as well as the adjoining structures. Numerical modeling is found to be an effective tool in understanding the behaviour of the ground under various conditions. Since the ground surface is inherently discontinuous, discrete element method (DEM) is adopted in this study to understand the mechanical behaviour of the ground. In this method, each particle is modeled independently and this grain scale modeling is used to understand the mechanical behaviour of the soil/rock masses. Here, a three dimensional ground surface is modeled in a weak ground and a tunnel of circular profile is excavated inside the modeled ground surface. Studies are done on cases like tunnels without lining and tunnels with lining. Studies are also done on a twin tunnel in the same model to estimate how the response of the ground changes under the same situations. The variation of the stresses and strains indicate that the lining has considerably improved the stability and reduced the deformations around the tunnels.*

1.0 INTRODUCTION

Tunnels play a significant role in the modern world due to lack of space. The stability of tunnel is one of the most important subjects in the tunnel constructions. The stability of a tunnel basically depends on the surrounding ground as well as the presence of the adjoining structures. With new technologies like using Tunnel boring machines and ground improvement techniques, tunnels can be constructed in weak ground, soft/hard rocks depending on the requirement. Once a tunnel has been excavated the surrounding ground becomes loosened reducing its stability. In such cases, linings and reinforcement play a very significant role in providing additional support to the already loosened ground mass. In this particular study simulations are performed in which an unlined tunnel and tunnel with lining are excavated in soft ground. Similarly simulations are done on a twin tunnel which is unlined as well as lined.

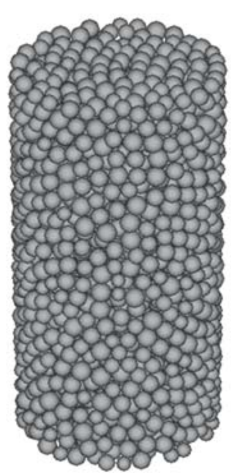
The discrete element method is adopted in this study to understand the various aspects of the stress distribution during tunneling. In this method the system is modeled as a conglomeration of particles which are discrete in nature and interact only through the contacts. One of the major advantages of this modeling is that the discontinuous nature of the medium or the presence of joints/ fissures can be easily accounted for. In order to account for the strength of the cementing material which binds together the particles in the case of weak ground, contact bonds are employed. This idea is based on the bonded particle model suggested by Potyondy & Cundall [1] wherein the rock mass is represented as a dense packing of circular or spherical particles which are bonded together at their contact points. These contact bonds are breakable in nature and the macro behaviour of the ground as a whole depends on the strength of these bonds. Since in this study weak ground is simulated the contact bond strength used is less. The stability of an underground structure like tunnel is very important since any ground movement associated with tunnel will result in unexpected catastrophes. A number of experimental and numerical studies have been by conducted by Wu & Lee [2], Choi et al [3], Rowe & Kack [4], Lee & Rowe [5] to understand the ground settlement above tunnels. Funatsu et al [6] have conducted 2-D DEM studies to understand the effect of lining and reinforcement on the stability of tunnels. Tannant & Wang [7] used bonded assemblies to simulate direct tension and block punching tests on tunnel liner materials to understand the effect of liner using 2D-DEM. They showed that the liner had minimal impact on fracture propagation and was able to retain the fractured rock in place. The study also reported that the presence of liner helped to control/reduce the displacements around the tunnel. In order to model tunnel excavation in soft ground, Kasper & Meschke [8] developed a 3D finite element model which was successful in reproducing the stresses and strains in the soil. Limited number of studies has been done using 3-D DEM to understand the effect of lining and reinforcements on the horizontal and vertical stress distributions and strain distribution around underground structures like single tunnels or twin tunnels. This paper presents a discrete element approach towards the stress and strain distributions around a single circular tunnel as well as a twin tunnel.

2.0 OVERVIEW OF DISCRETE ELEMENT METHOD

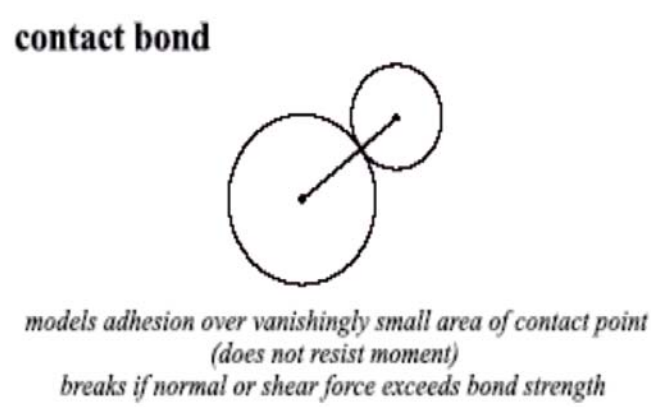
Cundall [9] pioneered the discrete element method (DEM) to analyze the rock mechanics problems considering the discrete nature of particles. The discrete nature results in the transferring of forces through the contacts between the particles. This violates the continuum assumptions and hence the constitutive behaviour and the deformation modes cannot be properly analyzed by continuum methods. DEM is an explicit finite difference method wherein the interaction of the particles is monitored contact by contact and the motion of the particles modeled particle by particle. DEM is following the alternate application of a force–displacement law at the contacts and Newton's second law to the particles. Using Newton's second law, the translational and rotational motion of each particle is calculated whereas the force–displacement law is used to update the contact forces arising from the relative motion at each contact. In this paper, the discrete nature of the granular materials is modeled using the three dimensional particle flow code [10] which is based on DEM.

3.0 PARAMETRIC STUDY

One of the most important requirements for numerical simulations is the proper selection of the material properties. The accurate modeling of the sample lies not only in the geometry of the model but also in the selection of the appropriate microparameters required for the model. In this study, to arrive at the material properties of a weak ground having minimum cementation, a series of axisymmetric triaxial tests are conducted on a cylindrical assembly whose height to diameter ratio is 2:1. The tests were conducted at a confining pressure of 1MPa. The assembly (Figure 1a) consists of 2700 spherical particles which are bonded together by contact bond (Figure 1b). The size of the particles used for the simulation varied from 0.075 m to 0.1 m. Figure 2 indicates the variation of uniaxial compressive strength of the sample with contact bond strengths at various confining pressures. The strength of the contact bond is significant as the overall strength of the material depends upon it. Higher the contact bond strength, higher will be the compressive strength. But in all the cases there is a linear variation in uniaxial compressive strength with contact bond strength. In this study contact bond strength of 20 kN is taken such that the sample is having a uniaxial compressive strength of nearly 2 MN. The variation of Young's modulus with different values of contact bond strength is presented in Figure 3(a). The contact bond strengths were varied from 10kN to 50 kN. At lower values of normal stiffness, the variation in the Young's modulus with the contact bond strength is almost insignificant. But at higher stiffness of 400 MN/m and 800 MN/m, it can be seen that there is a steep increase in the modulus value at lower contact bond strengths. Figure 3(b) presents the variation of Young's modulus value with different values of spring stiffness. The stiffness was varied from 10MN/m to 800 MN/m. The contact bond strengths for all the simulations have been varied from 0.01 MN to 0.05 MN. At higher contact bond strengths, however the modulus value remains more or less the same. Hence in order to represent a sample of uniaxial compressive strength of 2 MN, contact bond strength of 20 kN and particle stiffness of 100 MN/m is adopted in this study. In order to



(a) Cylindrical assembly



(b) Contact bond representation (Itasca PFC3D Manual)

Fig. 1 : Sample used for axisymmetric tests

understand the variation in angle of internal friction with respect to contact bond strength, the procedure suggested by ISRM [11] viz. "individual test criteria" is used. The peak axial stress values at various confining pressures are obtained and plotted against the corresponding confining pressures. This peak strength envelope can be represented according to the equation $\sigma = mp + b$, where ó is the axial stress, m is the slope of the envelope, p is the confining pressure and b is the ordinate. Knowing these values, the angle of internal friction φ is calculated as $\varphi = \sin^{-1}[(m-1) / (m+1)]$. Figure 3(c) shows the variation of angle of internal friction with contact bond strength. This indicates that the angle of internal friction at various contact bond strengths is almost the same with very little fluctuations. Hence corresponding to a contact bond strength of 20 kN, a value of 0.45 is taken as inter particle friction.

4.0 NUMERICAL MODELLING OF THE ASSEMBLY

A volume with dimensions 25m x 10m x 25m was modeled as shown in Fig. 4(a). This assembly was modeled using a total of 51000 particles. Table 1 shows the properties of the particles used for the simulation for both single and twin tunnels. The particles are allowed to settle down under gravity. After applying the gravitational force, the system is subjected to a confining stress of 0.8 MPa corresponding to the overburden pressure. This helps to develop an isotropic loading condition within the modeled assembly. The assembly is shown is Fig. 4 and a small rectangular portion in the centre is marked for showing the contact force and displacement distributions. The enlarged view of the contact force and displacement distribution of this isotropically consolidated assembly is shown in Figs. 4(b) and 4(c) respectively. These figures indicate that the contact forces and the displacement distributions are uniform for the small element considered. In order to excavate the tunnel, first a collection of particles which lie in the required location of the tunnel is identified. Once the particles are identified, these particles are removed/ deleted from the model. Traction/contact force due to removal of the particles will initiate deformation around the opening. The tunnel is excavated stage by stage following which the lining is installed immediately in the case of lined tunnels. Fig. 5(a) represents the model assembly with single tunnel and 5(b) represents the model assembly with twin tunnels. The shape of the

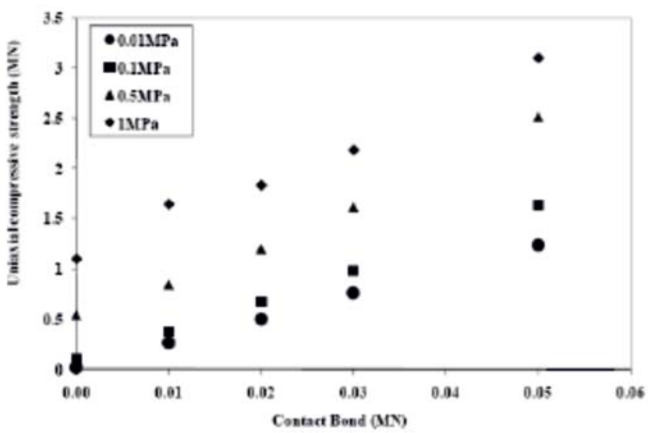


Fig 2 : Uniaxial compressive strength vs contact bond

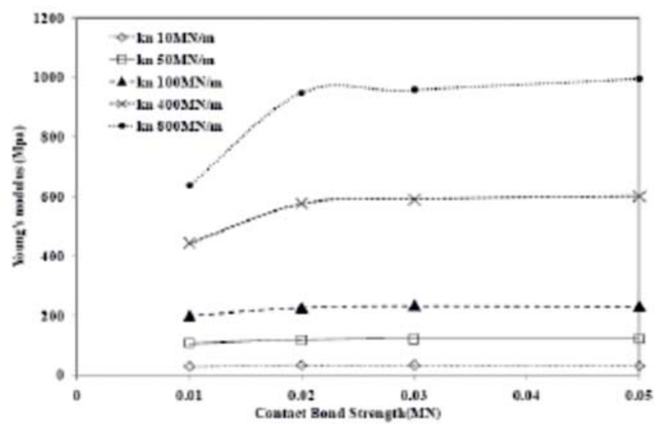


Fig 3(a) : Young's modulus vs contact bond strength

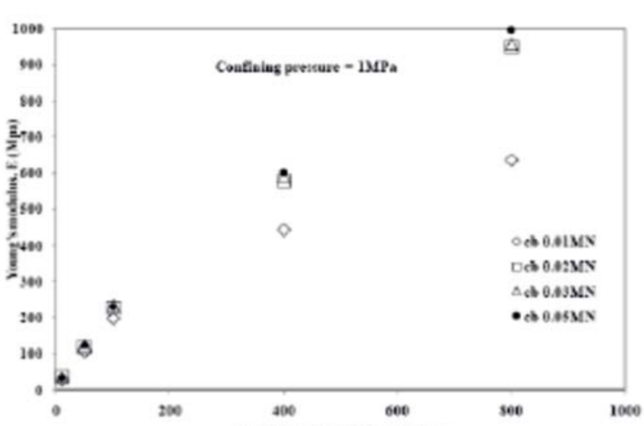


Fig 3(b) : Young's modulus vs spring stiffness

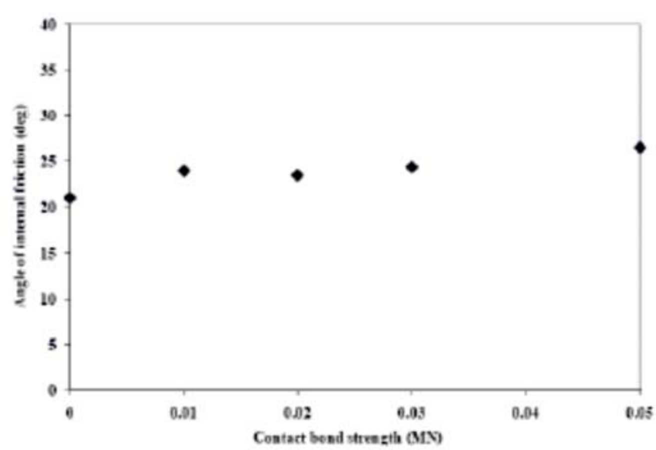
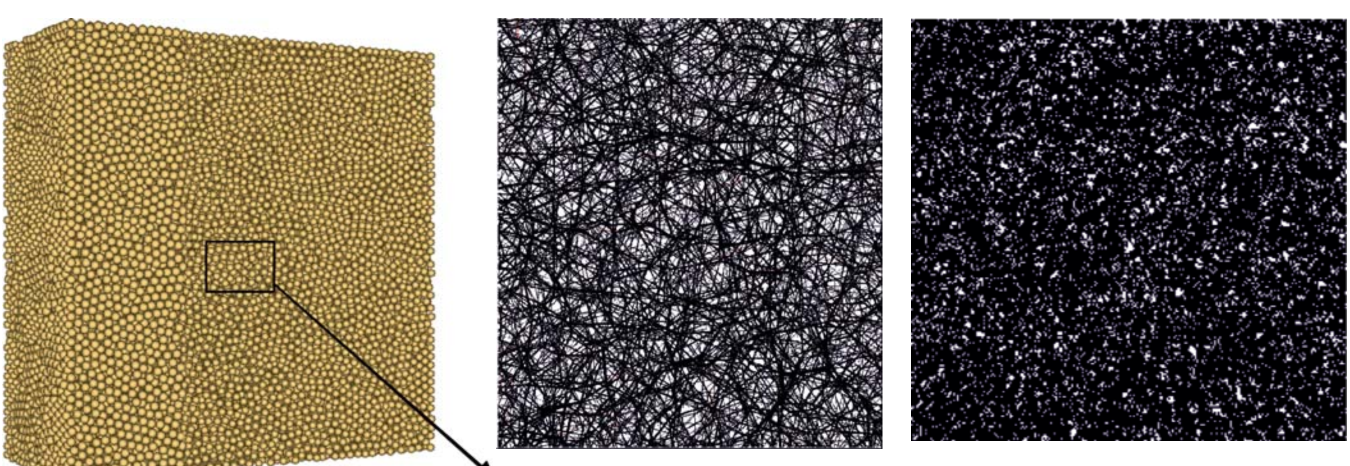


Fig 3(c) : Angle of internal friction vs contact bond strength tunnel cross section is termed as profile. The selection of a particular profile depends on several parameters like cost, easiness in construction, maintenance, risk etc. The most popular circular profile is used in this study for both the system of tunnels. The single tunnel is having a radius of 3 m and is excavated in the soil mass at a depth of 18 m from the ground surface. For twin tunnels, the radius of each tunnel was taken as 1.5 m and at the same depth as that of the single tunnel. Contact bonds with very low strength are installed within the sample to represent the weak ground. The effect of the excavation of the tunnel is studied with respect to the contact force, displacement distribution, circumferential stress and horizontal stress. The variation is studied for different cases viz. (a) the tunnel is left unlined/without any support and (b) tunnel is lined. The external load is distributed through the contacts and the distribution of contact force for single tunnel and twin tunnel is shown in Fig 6. The close up view in Fig. 6(a) indicates that the unlined tunnel has completely collapsed. It can be clearly observed that there is a significant reduction in the contact force (Figures 6b, 6d) once the tunnel is excavated leading to the ultimate failure of the tunnel in the case of single tunnel as well as twin tunnel. The displacement distribution shown in Figs. 6(c) and 6(e) shows the ground movement due to the excavation of the tunnel. It can be seen that the entire ground surrounding the tunnel has been collapsed. But if the tunnel is lined as in Fig. 7, it can be seen that the system is much stable when compared to that of an unlined tunnel. Moreover, the twin tunnel is more stable compared to the single tunnel and also the deformations are much less when compared to that of a single tunnel. This underlines the fact that twin tunnel will provide a better configuration and stability when compared to that of the single tunnel under the same loading and material properties.

Table 1 : Properties of the particles used

Property	Value
Normal stiffness of particles	100 MN/m
Shear stiffness of particles	100 MN/m
Wall stiffness	1000 MN/m
Density of particles	2600 kg/m^3
Normal Contact bond strength	20 kN
Shear Contact bond strength	20 kN
Porosity	0.4
No of particles before excavating tunnel	51000
Inter particle friction	0.45

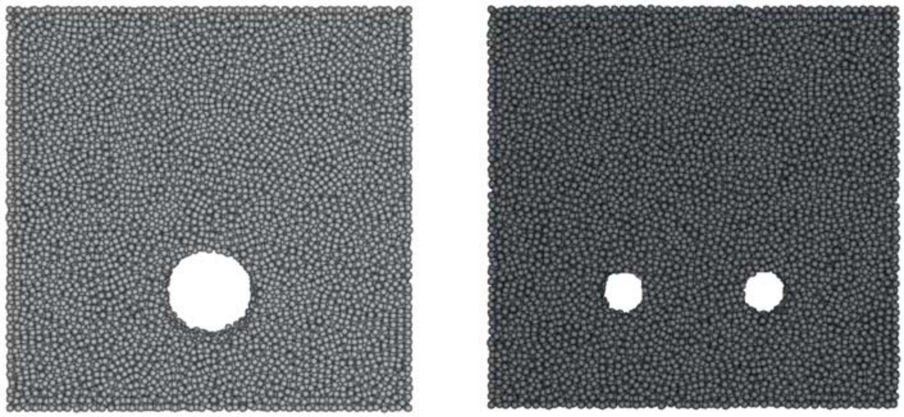


(a) Sample (25m x 10m x 25m)

(b) Contact force distribution

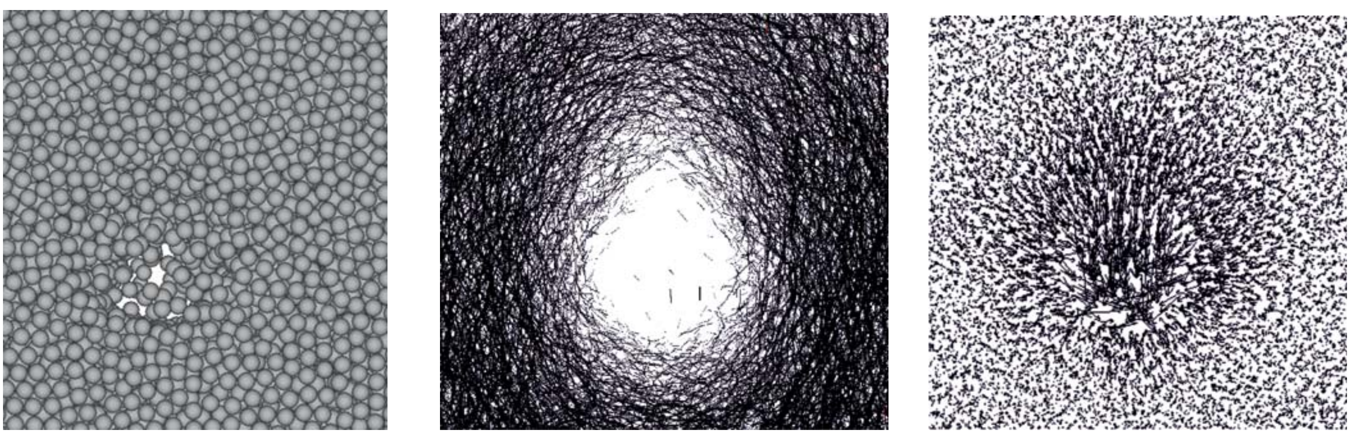
(c) Displacement distribution after isotropic compression (b & c **Enlarged view** of the central portion marked)

Fig. 4 : Initial stress state before the excavation of the tunnel



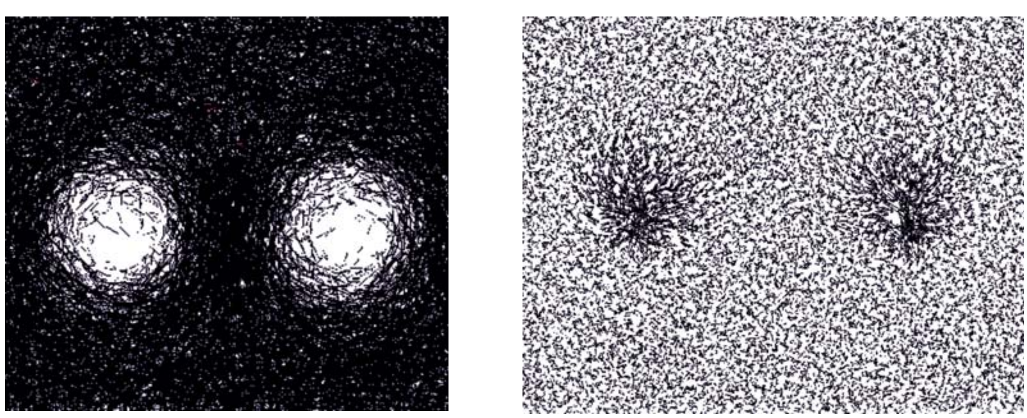
(a) Single tunnel (b) Twin tunnel

Fig. 5 : Excavated tunnels



(a) Unlined tunnel (b) Contact force distribution (c) Displacement distribution

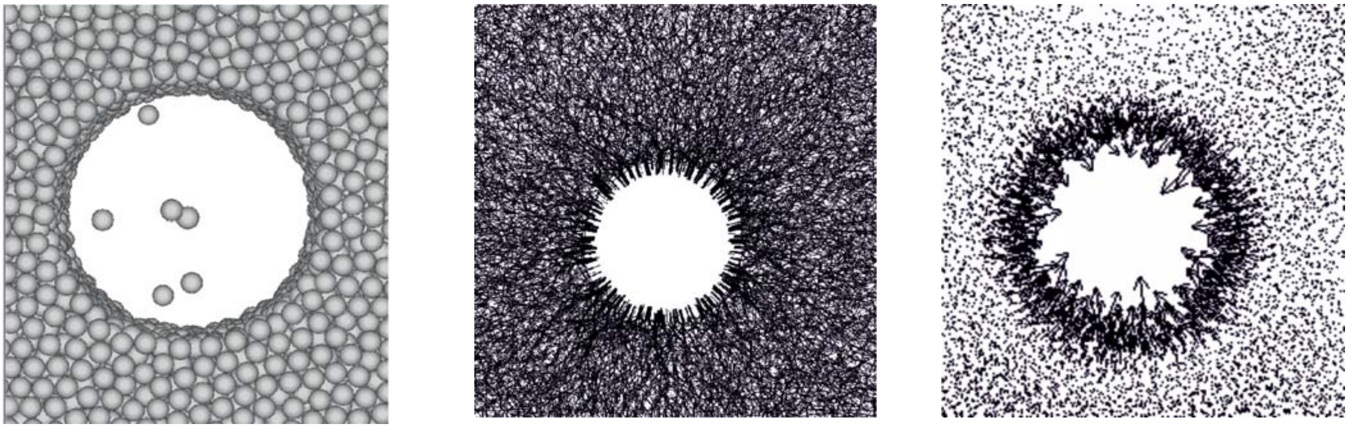
(i) Single tunnel



(d) Contact force distribution (e) Displacement distribution

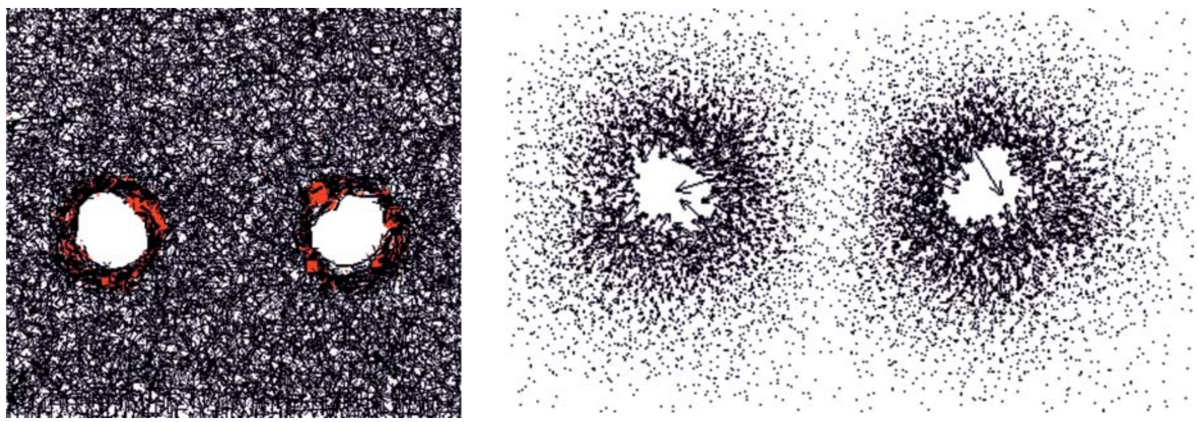
(ii) Twin tunnel

Fig. 6 : Stability of Unlined tunnel **(Enlarged view near the tunnel)**



(a) Lined Tunnel (b) Contact force distribution (c) Displacement distribution

(i) Single tunnel



(d) Contact force distribution (e) Displacement distribution

(ii) Twin tunnel

Fig. 7 : Stability of Lined tunnel **(Enlarged view near the tunnel)**

5.0 DISTRIBUTION OF STRESSES AND STRAINS AROUND THE TUNNEL

The stresses and strains developed in the assembly due to the excavation of the tunnels were measured by defining the measurement spheres. Measurement spheres (Fig. 8) help to measure various parameters namely porosity, stress, strain rate, coordination number etc. in an assembly over a specified spherical volume such that it represents a large number of particles. As the tunnel has a circular cross-section it represents an axisymmetric problem. The stress tensor obtained from the measurement spheres were in the Cartesian coordinate system. This is transformed to the local coordinate system corresponding to the direction in which the stresses are required and the circumferential stresses around the tunnel were calculated at different inclinations. Figure 9 indicates the circumferential stress distribution for the tunnels and it can be seen that the values are higher for the structure with lining. Moreover it can be seen that the stresses are almost uniformly distributed around the tunnel. The presence of lining provides a strong support and hence restricts the movements of particles. This results in a stable configuration of the system when compared to unlined systems. The stress distribution of an unlined tunnel indicates that the stresses are very low near the tunnel. However the progress of failure in the case of unlined tunnel is almost same in all the direction. This can be seen from Fig. 6 where the tunnel collapses from all sides, more being from the top. The variation of stresses on tunnel with lining show comparatively higher values and are almost uniform along all the directions indicating that the ground is stable. Fig 10 represents the distribution of vertical strains for the single tunnel and twin tunnel along the height of the ground mass considered. It can be seen that the vertical strains are very much higher for unlined tunnels compared to that of lined tunnels. It can be seen that the maximum vertical strains in the case of unlined single tunnel was about 9% whereas for the unlined twin tunnel it is about 6%. The vertical strains were extracted along the centre line of the model and is plotted with respect to the depth from the ground surface. It should also be noted that the strains depend on the spacing between the tunnels also. Similarly in the case of lined tunnels also the amount of strain experienced is less for twin tunnels. This underlines the fact that the twin tunnels will reduce the overall disturbance of the surrounding ground. The distribution of vertical stresses is shown in Fig. 11. The variation of vertical stresses show that the lined twin tunnels are having maximum vertical stresses just above the crown compared to other sections. In Fig. 12, the variation of tangential stresses is plotted. It can be seen that the tangential stresses increases slightly above the crown of the tunnel. This may be due to the fact that just above the crown of the tunnel, the soil mass is highly displaced leading to a loose packing and beyond that the packing is dense.

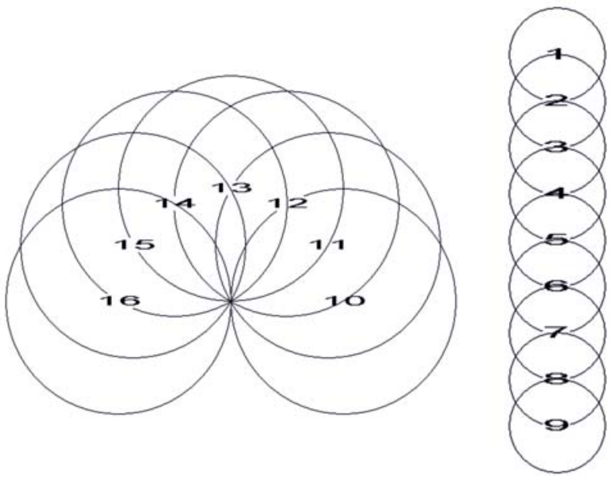


Fig. 8 : Measurement spheres to measure porosity, stress, strain rate etc in a spherical volume

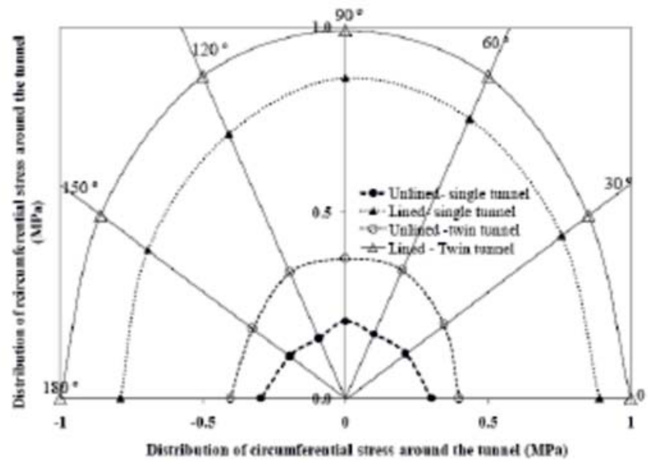


Fig. 9 : Distribution of stresses around the tunnels

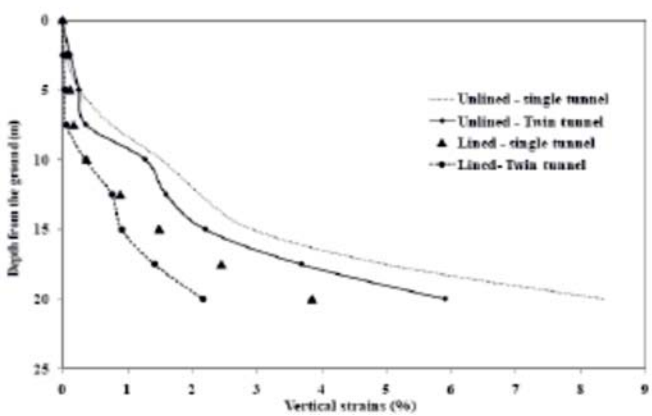


Fig. 10 : Distribution of vertical strains along the centerline of the model vs depth

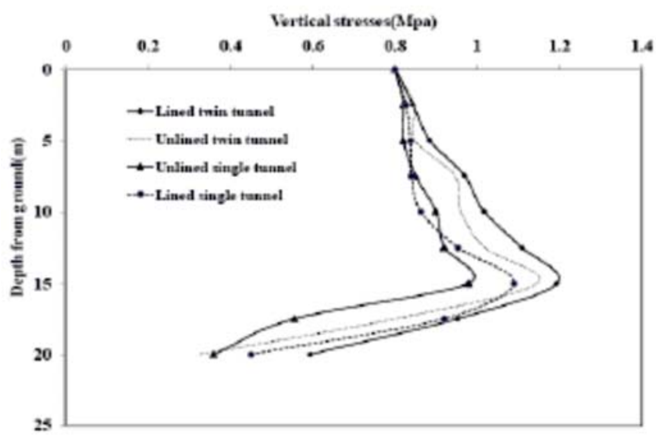


Fig. 11 : Distribution of vertical stresses

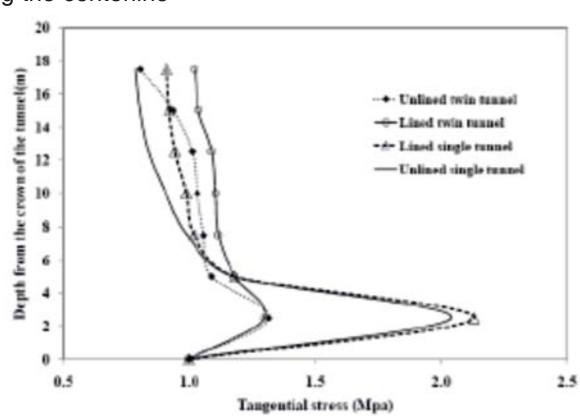


Fig. 12 : Distribution of tangential stresses

6. CONCLUSIONS

The evaluation of the response of the tunnels using discrete element method indicates that the presence of lining decreases relaxation around the tunnel. The increase in the contact force around the tunnel due to the presence of lining results in arching which leads to a more stable tunnel assembly. The displacement distribution clearly shows how the unlined tunnel collapses and the same can be seen from the contact force and variation of vertical strains. When lining is present it increases the confinement of the surrounding soil and forces it to act as a single mass. The displacement and contact force distributions for a lined tunnel clearly implies a stable configuration for lined tunnels. Also the assembly of twin tunnels is more stable with less displacement or strains compared to single tunnels and this emphasizes the significance of twin tunnels over a single tunnel.

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FORTHCOMING ACTIVITIES OF INDIAN NATIONAL GROUP OF ISRM

Seminar on "Geotechnical Challenges in Water Resources Projects", 19-20 January 2012, Dehradun

Execution of multipurpose water resources projects located in complicated geological settings is a challenge to engineers. Despite tremendous all round advancement in technologies, there is still scope to learn and know more about the geotechnical challenges faced during excavations and its implications on design as well as execution. Both design and construction engineers would always be keen to know what advancements are taking place in their respective fields to face such challenges and utilize the knowledge for most economical and safe design for construction of a project.

These projects require application of modern principles of rock mechanics, which warrants deliberations and collaboration to facilitate flow of appropriate technology to enable successful implementation of such projects under a time-bound programme in a cost-effective manner, conforming to environmental requirements.

Safety during tunnel and underground construction is of paramount importance not only to the stability of structures but also to ensure safety of the work force. Safety in tunnels and cavities may be endangered due to sudden rock falls, geological failures, collapse of underground opening or a part of opening, etc.

The objective of the proposed Seminar, being organized by the Indian National Group of International Society for Rock Mechanics and the Central Board of Irrigation & Power and, is to provide a forum for the design and construction engineers to discuss the solutions to various geological and geotechnical surprises encountered during the execution of various River Valley Projects and use the knowledge for safe and economical design and construction of such projects.

DETERMINATION OF STRESS ORIENTATION AND MAGNITUDE IN A DIRECTIONAL WELL BORE, CASE STUDY: MANSOURI-54 WELLBORE OF SOUTHWEST IRANIAN OILFIELD

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Abstract *: Knowledge of insitu stress tensor is critical in developing petroleum fields, mineral resources and other underground excavations. Since this research is focusing on stress determination in a directional wellbore, based on available data, the well-54 from the Mansouri field was selected as a case study. The wellbore is located in the Zagros basin in south-western of the Iran. With ultrasonic borehole image log (UBI), the orientation of the minimum horizontal stress (S_h) was determined using borehole breakout method. Then, the magnitude of vertical stress (S_v) was calculated by integrating density formation from surface to the given depth. In fact, magnitude of S_h was determined by generalizing Breckels and van Eekelen's expressions (Fjaer, et al. 2008). At any given depth the maximum horizontal stress (S_H) constrained by Anderson's faulting theory and Coulomb faulting theory for an assumed coefficient of friction. In the present study, the stress gradients have indicated that the stress regime corresponds to the strike slip faulting condition ($S_x \equiv S_H > S_z \equiv S_v > S_y \equiv S_h$). It was observed that S_h direction corresponds to the Zagros thrust belt and optimum orientation of drilling the wellbores to minimize wall collapse in this area is located on NE-SW direction.*

Keywords: *Insitu stress, Mansouri-54 wellbore, Anderson's faulting theory, Image log, Zagros thrust belt.*

1.0 INTRODUCTION

A practical engineering solution to optimal mud weight determination, stable trajectories and casing set points problems requires the accurate knowledge of stress state at depth (Willson 1999). To determine insitu stresses, Zoback et al. (1993) and Brudy et al (1997) for the first time performed a comprehensive project using an integrated stress measurement strategy (ISMS). In the same fashion, Reynolds et al. (2006) tried to constrain the insitu stresses at Cooper- Eromanga basin in Australia. They concluded that the insitu stress is the direct outcome of complex interactions between tectonic stresses and convergent plates. In this research, the orientation and magnitude of insitu stress in the Mansouri-54 wellbore is constrained using Anderson's frictional faulting. In the drilling process, usually two types of common wellbore instability occur: wellbore breakout and drilling induced tensile fractures (DITF). In fact, the borehole breakouts are stress-induced enlargements of the wellbore cross-section (Bell 1990).

When a wellbore is drilled, materials removed from the subsurface no longer support the surrounding rock hence, stresses become concentrated in the surrounding rock (i.e., the wellbore wall). In other words, the borehole breakout occurs once the stresses around the borehole exceed compressive strength of the rock around the borehole wall (Zoback et al. 1985). The wellbore breakout is caused by the development of intersecting conjugate shear zones which tend to spalling of the wellbore walls (Figure 1). Around a vertical borehole, the stress concentration (ratio of rock compressive strength to maximum principal stress) is greatest and positive in the direction of S_h while it has minimum and minus magnitude in S_H direction. Hence, the long axis of borehole breakouts are oriented approximately perpendicular to the S_H direction (Plumb and Hickman 1985).

Figure 1 shows creation of borehole breakout along the S_h that indicated as $\sigma_{h\min}$. The borehole breakouts are interpreted typically from acoustic image log data using the borehole radius (or travel time) image in combination with images of the reflected amplitude. In fact, the drilling-induced fractures can only be observed on image logs. Both natural and drilling-induced fractures are poor reflectors of acoustic energy (Tingay et al. 2008). Using borehole breakout and Anderson's theory, Wiprut and Zoback (2000) constrained the insitu stresses in Visund field. They determined the stress region according to the variation of the rock compressive strength in five wellbores. In most of the wellbores, there were no tensile fractures. Sometimes transverse drilling induced fractures occur when the axial stress falls below the rock tensile strength.

Nelson et al (2005), utilizing Anderson's frictional faulting theory and transverse drilling induced fractures, constrained insitu stresses in West Tuna area of Gippsland basin in Australia. With respect to the case study of this research the reservoir cap-rock in the Mansouri-54 wellbore is located in 2335m (True Vertical Depth) and the final depth is about 3485m (TVD). Here, the wellbore breakout data was obtained from ultrasonic borehole imager (UBI). It can be realized that the azimuth of S_h is $N135 \pm 10S$, which corresponds to the Zagros thrust belt direction, approximately.

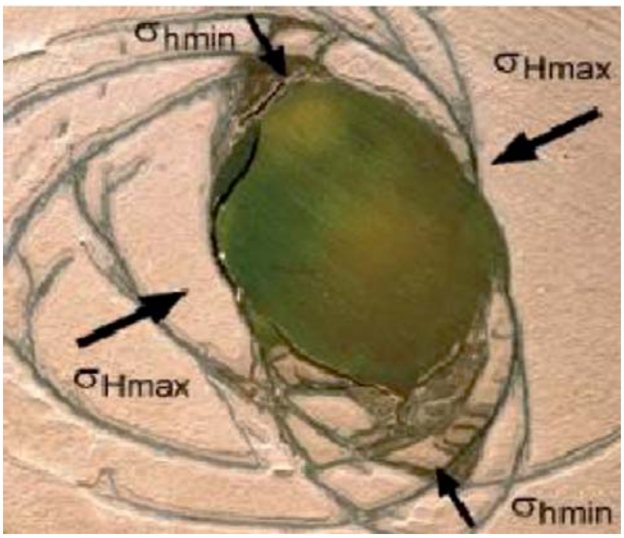


Fig. 1 : Breakout occurs along the minimum horizontal stress (S_h) on the other hand it forms perpendicular to direction of the maximum horizontal stress (S_H) (Tingay et al. 2008).

2.0 GEOLOGY OF MANSOURI-54 WELLBORE

The Zagros fold-thrust belt stretches around 2000 km from south-eastern Turkey through northern Syria and Iraq to western and southern Iran (Alavi, 2004). Structurally, the Zagros basin is placed in the north of the Arabian plate. The case study is located at Dezful embayment of the Zagros basin in Southwest of Iran (Figure 2). Sequential formations of the case study have been showed in Figure 2. As shown, studied formations of the well include: Pabdeh, Gurpi, Ilam and Sarvak. The Zagros basin is part of the Tethys Ocean and is one of the most important petroleum reservoirs in the world (Alavi, 1994). The geological history of this basin includes a long time subsidence and deposition which is interrupted by a short-term uplift. Its folding process began during Miocene and Pliocene anticlines and continued until now hence, forming a long anticline (Mottie 1995). These anticlines constitute most of oil traps in this basin. Mansouri oilfield is located in the northern Dezful subsidence at 60 Km south of Ahvaz. It is surrounded by the Ahvaz field from the northwest, Abteimoor field from the west and Shadegan field from the northeast. At oil-water contact surface, the Mansouri field is 39 Km long and 3.5 km wide.

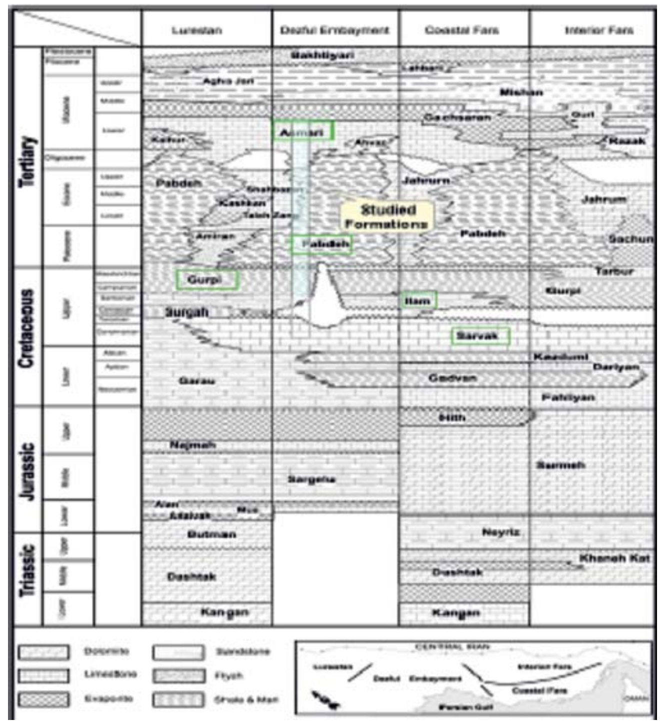
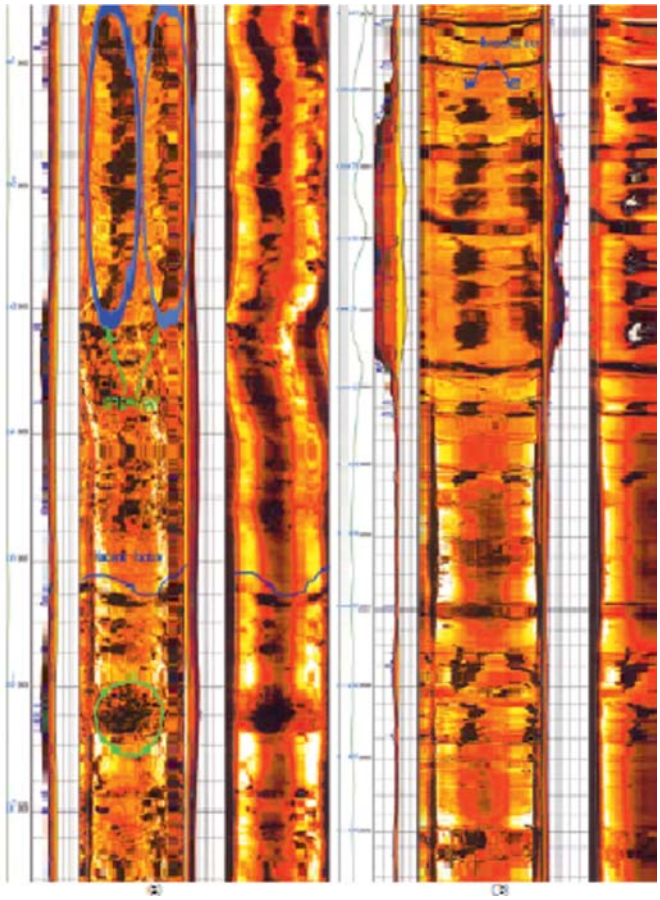


Fig. 2 : Mesozoic–Cenozoic stratigraphy correlation chart of the Zagros basin (Khoshbakht 2009).

3.0 INSITU STRESS ORIENTATION IN MANSOURI-54 WELLBORE

So far as the depth of oil and gas exploration is concerned, insitu stress orientations manifest by a number of phenomena. These include fault activity, borehole breakout, core discing, center line fracturing in cores, drilling-induced fractures and natural and induced earthquakes (Bell, 1996). Breakouts and drilling-induced fractures are the most commonly encountered phenomena (Bell, 2003). Here the wellbore breakouts are illustrated using Ultrasonic Televiewer Imager (UBI) log which have been obtained from 3650 m to 3695 m depth of the wellbore. Figure 3 highlights different instability phenomena in the wellbore. As shown, the wellbore breakouts appear as dark bands of low reflectance on opposite sides of the wellbore at the depth interval of XX1-XX3 and XX28-XX31 in Figure 3(a) and Figure 3(b), respectively. According to observed breakouts at the Mansouri-54 wellbore, it can be inferred that azimuth of S_h is about (N135 10S), with orientation of NE-SW direction. Since failure and collapse would lead to the high circumferential stress ($\sigma_{\theta\theta}$), the best direction



ρ_b

Fig. 3 : Appearance of breakouts on UBI in Mansouri-54 wellbore shows minimum horizontal stress direction (a-Depth: 3660-3678; b-Depth: 3679-3690)

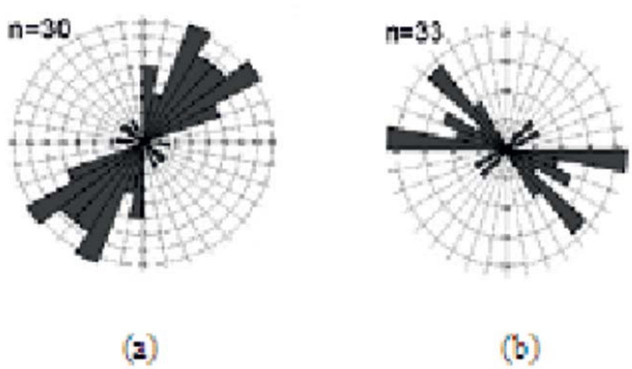


Fig. 4 : Horizontal stress orientation in Zagros belt according to Navabpour et al. (2007). (a) The maximum horizontal stress orientation, (b) The minimum horizontal stress orientation.

of drilling in order to minimize circumferential stress is S_H direction (NE-SW). Acknowledging our results, Navabpour et al. (2007) carried out a systematic analysis of the stress orientation in the Zagros belt. They analyzed the acquired brittle tectonic data from more than ninety sites which revealed a variety of stress regimes (strike–slip, compressional, and tensional types) and stress directions. Due to proximity to the surface, their observation was based on the strike-slip tectonic regime. They determined that S_H was directed to NE-SW (Figure 4(a)) and S_h was oriented to NW-SE (Figure 4(b)).

4.0 VERTICAL STRESS, PORE PRESSURE AND MUD PRESSURE

In flat terrain the vertical stress at a proposed depth can be assumed integrating the material densities from surface to depth i.e.:

$$S_V = \rho_w g z_w + \int_0^z \rho_b(z) g dz \quad \text{...(1)}$$

Where, (*z*) and ρ_w are the rock and water densities, respectively. In offshore areas, to calculate the vertical stress, the water depth (z_w) needs to be considered. In the current research, the pore pressure is based on hydrostatic pore pressure (Zoback et al. 2003). Since no information on Equivalent Circulating Densities (ECD) exists for the wellbore, static mud weights have been used to calculate S_H. It is observed that due to the circulation of the drilling mud, the equivalent circulating densities are larger than static mud weights. However, the static mud weights may over-estimate the magnitude of the horizontal stresses. In this wellbore, the static mud weight was determined to be equal to 69 pcf (1105 kg/m^3) for the Pabdeh, Gurpi and Ilam formations and 76 pcf (1216 kg/m^3) for the Sarvak formation. Based on its density logs, the average rock density of this particular wellbore is about 2.55 gr/cm^3. Table1 indicates the calculation of the hydrostatic pore pressure (P_{fn}) and the vertical stress at formations.

Table 1 : The vertical stress and hydrostatic pore pressure of formations

Formation	Pabdeh	Gurpi	Ilam	Sarvak
H(m)	3012	3209	3377	3485
σ_v (MPa)	75.72	81	84.5	87.4
P_{fn} (MPa)	29.52	31.4	33.1	87.4

5.0 MINIMUM HORIZONTAL STRESS

The S_h magnitude in petroleum basins is commonly determined using hydraulic fracture (HF) and leak-off tests (LoTs). Due to lack of HF test in the proposed case study, experimental expressions of Breckels and Van Ekelen are used to determine S_h. They used hydraulic fracturing data from whole regions in US Gulf Coast, and derived relationships between horizontal stress and depth. For the US Gulf Coast, Breckels and Van Eekelen presented the following relations (Fjaer et al. 2008, p.106):

$$S_h = 0.0053^{H1.145} + 0.46(P_f - P_{fn}) \quad \text{if } H<3500 \qquad ...(2)$$

$$S_h = 0.0264H - 31.7 + 0.46(P_f - P_{fn}) \quad \text{if } H>3500 \qquad ...(3)$$

Where S_h is the minimum horizontal stress in MPa, H is the depth of interest in meters, P_{fn} is the hydrostatic pore pressure in MPa and P_f is the formation pore pressure from the Repeat Formation Test (RFT) in MPa. Repeat Formation Tester is a well test tool has been introduced in 1975 to repeatedly measure formation pore pressure in the well (Ireland et al. 1992). In this research, the above Equations have also been generalized to other regions involved in the Mansouri-54 wellbore. In fact, the relations found to be useful for several case studies. Regressions were carried out between HF real data of S_h in Table 2 and Equations (2) and (3). For the current case study, merely two HF tests were done and implied in the regressions. The pore pressure formation was obtained through the RFT in the Mansouri-54 wellbore which is P_f=32.6 MPa in the Pabdeh, Gurpi and Ilam and P_f=41.5 MPa in the Sarvak formations. Figure 5 and Figure 6 highlight the regressions results at the depth of less than 3500m and H>3500 where correlation coefficient, R^2 are 0.9 and 0.97 respectively. However, it is possible to correct exponential coefficient of the Equation (2). Using this logic the coefficient was corrected and changed to 1.17. Eventually for the formations, S_h can be calculated using Equation (3) which is presented in Table 3.

Table 2 : The minimum horizontal stress from hydro fracturing test in different regions .

Wellbore location	Depth (m)	S_h from HF test (MPa)
Dullingari-8(Reynolds, 2006)	2800	56
Bulyeroo-1(Reynolds, 2006)	2794	57
Graveberg-1(Lund &Zoback, 1999)	2300	45
West tuna area(Lund & Zoback, 1999)	2500	51.25
Visund field (Wiprut & Zoback, 2000)	2830	53.2
Cajon pass (Zoback & Healy, 1992)	1850	37
Wellbore in China (Cui, 2009)	1269	25.3
Wellbore-6 (Wiprut &Zoback, 2000)	3720	71.5
Wellbore-8 (Wiprut &Zoback, 2000)	3560	67.5
KTB (Haimson & Chang, 2002)	3650	62.5
KTB (Haimson & Chang, 2002)	3700	63.25
KTB(Haimson & Chang, 2002)	3750	64.37
KTB(Haimson & Chang, 2002)	3800	65.5
KTB(Haimson & Chang, 2002)	3900	67.7
KTB(Haimson & Chang, 2002)	4050	71.12
KTB(Haimson & Chang, 2002)	4200	74.5
Mansouri- 54 wellbore	1595	19.56
Mansouri- 54 wellbore	2481	26.52

Table 3 : Magnitude of Sh in different formations of Mansouri-54 well.

Formation	Pabdeh	Gurpi	Ilam	Sarvak
S_h (MPa)	63.7	68.6	72.9	77.35

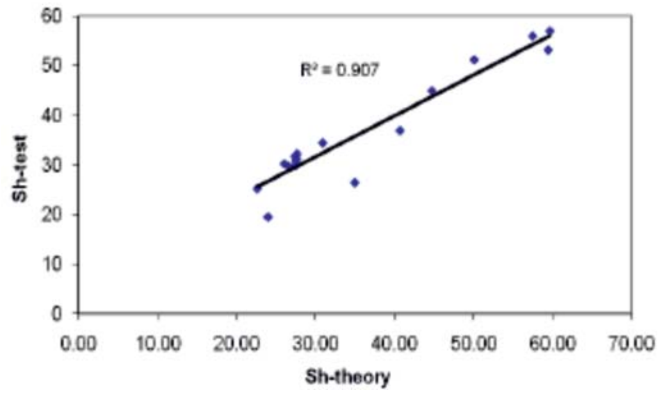


Fig. 5 : Correlation between S_h from hydro-fracturing (S_h-test) and experimental relation (S_h-theory) (H<3500) in the Mansouri-54 well.

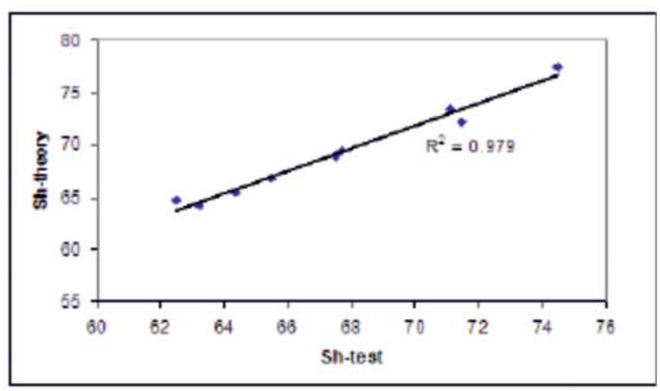


Fig. 6 : Correlation between S_h from hydro-fracturing (S_h-test) and experimental relation (S_h-theory) (H>3500) in the Mansouri-54 well.

6.0 MAXIMUM HORIZONTAL STRESS

The S_H can be estimated based on the methodology presented by Barton et al. (1988). They used breakout width and rock strength to calculate S_H. As such, by integrating this methodology to the stress polygon S_H can be obtained. In the petroleum industry, mechanical parameters of rock around the wellbore are sometimes calculated indirectly using Dipole shear Sonic Imager

(DSI). Figure 7 is a DSI log of this well which illustrates rock mass properties includes: uniaxial compressive strength (UCS), Young's modulus (E) and Poisson's ration (PR). In the first zone i.e., the Pabdeh formation UCS varies between 68 to 70 MPa. In the Gurpi formation, due to interbedded shale varies between 35 to 55 MPa. Since, Stonely wave which propagates from DSI tool attenuates and dissipates in permeable formations, an increase in its velocity decreases the Young's modulus (E) and UCS hence, it can be concluded that the enlargement of the wellbore seriously affects the rock properties. In the Sarvak zone, the UCS value varies between 100 and 120 MPa. The stress polygon is based on the Anderson's faulting theory and Coulomb frictional theory. Whereas observed fractures on UBI are breakout or compressional fractures, then the S_H is constrained in stress polygon by applying UCS values in these formations. Figure 8 and Figure 9 show constraining of S_H in stress polygon at the Pabdeh and Sarvak formations. As S_h was obtained in previous section, the stress polygon can be obtained here by drawing the vertical dashed line from S_h axis and constraining the S_H using horizontal dashed line. The above procedure can be repeated for other formations as well. The Mansouri-54 wellbore is the directional wellbore with azimuth of 25° to the north and 39° dip, horizontally. It is supposed to obtain stress around the wellbore in its direction. The transferred insitu stresses (S_x, S_y and S_z) along the wellbore can be determined by substituting α =25° and i=39° and insitu stresses into transforming expressions (Fjaer et al. 2008, p.146)

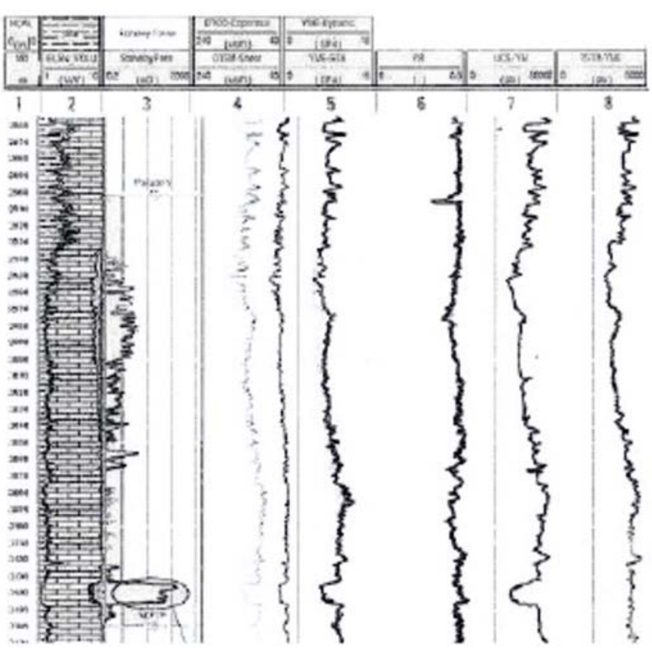


Fig. 7 : Dynamic rock mass properties in the Mansouri-54 wellbore.

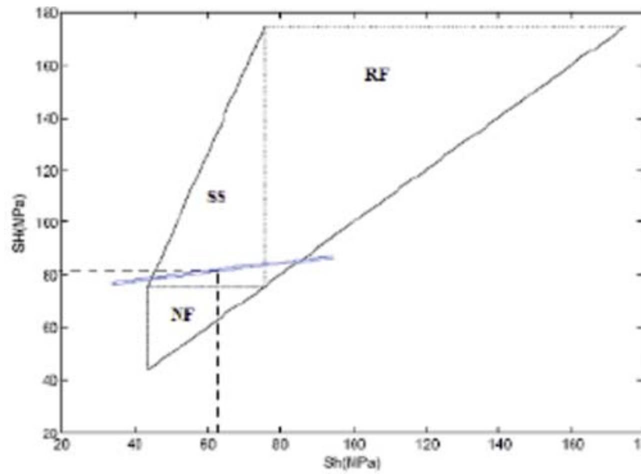


Fig. 8 : Stress polygon which describes magnitude of S_H in the Pabdeh formation at 3012 m.

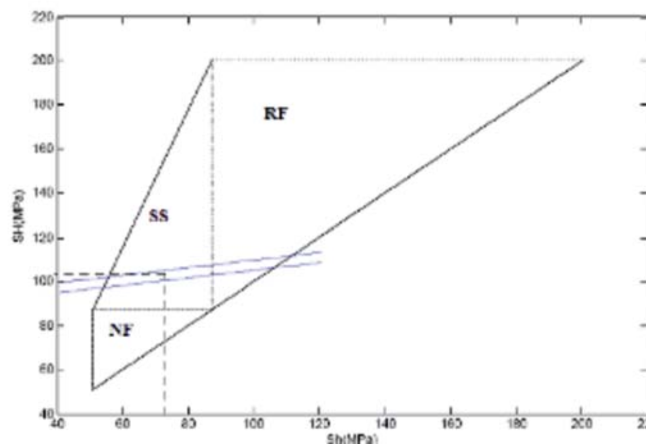


Fig. 9 : Stress polygon which describes magnitude of S_H in the Sarvak formation at 3485 m.

Stress gradient in formations of the wellbore has been showed in Figure 10. However, the difference between the mud pressure (P_w) and the pore pressure show that the wellbore was drilled overbalance. The overpressure balance (mud pressure minus pore pressure) was equal to 5MPa. According to Figure 10, stress gradients corresponded to the strike slip faulting condition stresses i.e. S_xa"S_H>S_za"S_v>S_ya" S_h. It should be noticed that by increasing the depth of the wellbore, the stress regime can be changed from SS to NF and vice-versa.

7. CONCLUSION

In this study, orientation and magnitude of insitu stresses were calculated in Mansouri-54 wellbore and hence, an optimum trajectory for drilling could be obtained. Here, the vertical stress for interval depth (3012 to 3485 m) was obtained by integrating rock density from surface to the depth. The minimum and the maximum horizontal stresses were calculated using practical relations and Anderson's frictional theory, respectively. At the same time, the DSI log was used to acquire the rock properties

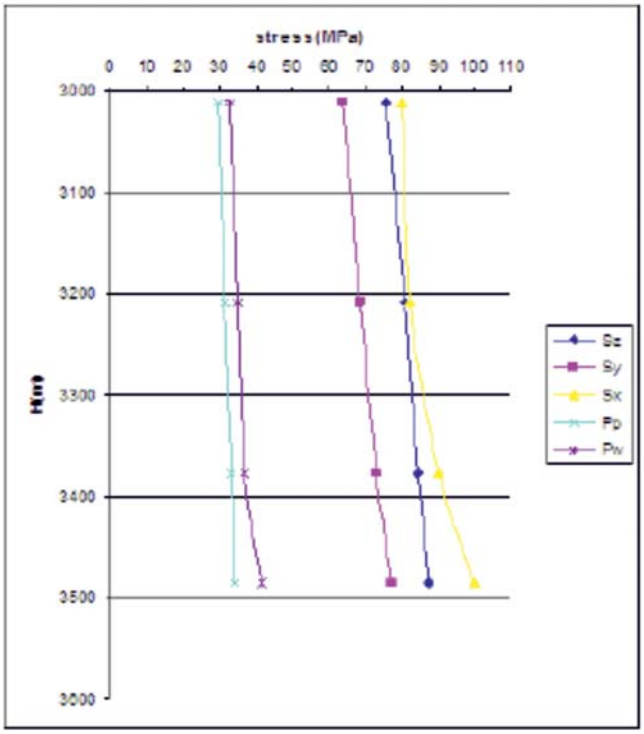


Fig. 10 : Insitu Stresses, mud pressure and pore pressure gradients around the Mansouri-54 wellbore

hence; it was assumed that the rock behaviour is isotropic, elastic and homogenous. By obtaining the stress regime in the wellbore direction (=25° and i=39°) it was demonstrated that the stress regime corresponds to the Strike Slip faulting (S_xa"S_H>S_za"S_v>S_ya"S_h). The optimum trajectory for stability during drilling is azimuth at which circumferential stress is minimized. Stable azimuth in the wellbore is S_H i.e. NE-SW. To minimize the circumferential stress, the overpressure balance was equal to 5 MPa.

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INDIAN NATIONAL GROUP OF ISRM

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Brief Profile of the Office Bearers of Indian National Group of ISRM

DR. HARI R. SHARMA - PRESIDENT

Dr. Hari R Sharma, born on April 03, 1934, graduated in Civil Engineering in 1956. Obtained 'Doctor of Engineering' (Dr. – Ing.) degree from Germany in 1964 and that of 'Doctor of Technical Sciences' (Dr. Techn.) from Norway in 1974.

Dr. Hari R Sharma has more than 54 years experience in Hydro Power. He has been associated with practically all major water resources / hydro power projects of India and neighbouring countries.

Dr. Sharma has evolved unique designs and has carried out several innovations in the field of Water Resources and Hydro Power Engineering. He has occupied very high positions in government of India including Member (Hydro), Central Electricity Authority & Ex-Officio Addl. Secretary to Govt. of India; Chairman, Nathpa Jhakri Power Corporation. He has been Advisor on Water & Power to Government of Mauritius for more than 12 years. He has served Australian Consultancy Firms Snowy Mountain Engineering Corporation and Hydro Tasmania Consulting as General Manager (Hydro & Tunnels) and Chief Technical Principal respectively.

Presently, he is Director (Design & Engineering), GVK Technical & Consultancy Services Pvt. Ltd.

- He has contributed more than 100 technical papers in national & International Journals / Conferences.
- He is widely travelled both in India and abroad including US, Russia, Japan, China and practically all European countries a number of times.
- He is Fellow of Indian National Academy of Engineering
- He has several Awards to his credit.
- He was nominated **International Engineer of the Year – 2008** by the International Biographical Centre of Cambridge, England.

DR. R.K. GOEL - VICE PRESIDENT

Dr. R.K. Goel postgraduated in Applied Geology from the University of Roorkee (now IIT Roorkee) in 1982. He obtained Ph.D. in Mining Engineering from VRCE (now VNIT), Napgur in 1995. Dr. Goel is working in Central Institute of Mining and Fuel Research (Erstwhile CMRI) since 1982 and presently holding the post of Scientist-G. His work areas are engineering geology and rock mechanics applications to tunnelling and underground space technology. During his 27 years of experience, he has worked on many tunnelling projects of Himalayan region and other parts of India. These include R&D and consultancy projects of Ministry of Defence, BARC, Ministry of Water Resources, DST and various other hydroelectric, railway and road tunnel projects. In addition to various research awards of IGS, CMRI, UOR (now IITR), he has received the highly recognized Ministry of Mines, Govt of India's National Mineral Award 2004 for his outstanding contributions in Engineering Geology. He has co-authored three books and published over 95 research papers in National and International Journal and Conferences. He is Editorial Board Member of Tunnelling and Underground Space Technology, a Journal of Elsevier Science Ltd. and Editor, Journal of Rock Mechanics and Tunnelling Technology. He is President, Indian Society for Rock Mechanics and Tunnelling Technology (ISRMTT), Roorkee Local Chapter and Vice President ISRM(India).

SEMINAR ON "GROUTING AND DEEP MIXING"

25-26 August 2011, New Delhi

A view of the dignitaries on dais during the Inaugural Session (L to R) Mr. V.K. Kanjlia, Secretary, CBIP, Dr. V.R. Raju and Mr. S.P. Kakran

Mr. S.P. Kakran lighting the lamp to mark the auspicious begining of the Seminar

Many methods for ground modification and improvement are available, including dewatering, compaction, preloading with and without vertical drains, admixture stabilization, grouting of several types, deep mixing, deep densification, and soil reinforcement. However, there have been rapid advances in the areas of deep densification (vibro-compaction, deep dynamic compaction piles, and explosive densification), jet and compaction grouting, deep mixing, and stone column systems in recent years. These methods have become practical and economical alternatives for many ground improvement applications. Many of these methods have been applied to increase the liquefaction resistance of loose, saturated, cohesion-less soils.

Keeping in view the importance of the aforesaid methods for ground modification and advances taking in the field, Indian National Group of International Society for Rock Mechanics with the support of the Central Board of Irrigation and Power (CBIP), Indian National Committee of International Tunnelling and Underground Space Association (TAI) and Indian National Committee on Large Dams (INCOLD) organised a Seminar on "Grouting and Deep Mixing" at CBIP Conference Hall, Malcha Marg, Chanakyapuri, New Delhi on 25 and 26 August 2011. The objective of the Seminar was to provide a forum for discussion and interaction amongst the participants to discuss potentially applicable ground improvement methods for civil works structures.

The Seminar was attended by 55 participants from 27 organisation/institutes from across the country and Bhutan.

The Seminar was inaugurated by Mr. S.P. Kakran, Member (D&R), Central Water Commission, on 25th August 2011 and the Inaugural Session was presided over by Dr. V.R. Raju, Managing Director, Keller Far East, Singapore.

Mr. S.P. Kakran in his inaugural address mentioned that complex ground situations are being encountered at many project sites requiring use of innovate ground improvement techniques. However, these techniques have their own merits and demerits and have to be selected carefully in view of their limitations. Jet grouting below coffer dams is now used quite often to arrest seepage in the water resources projects. He also mentioned that to keep pace with the rapid infrastructure development, we have to look out for advanced techniques which can provide technically superior, cost effective solutions.

Dr. V.R. Raju in his presidential address presented an overview of "Deep Mixing", and its utility to improve shear strength or bearing capacity and to reduce permeability

During the five Technical Sessions, following papers/case studies were presented and discussed:

- Deep Soil Mixing: Technique & Applications – *Dr. V.R. Raju, Keller Far East, Singapore*
- Restoration and Rehabilitation of Old Pagara Masonry Dam by Grouting Technique (A Case Study) – *Mr. Mahavir Bidasaria, Ferro Concrete Const. (India) Pvt. Ltd*
- Treatment of Unconfirmity Zone and Curtain Grouting in Foundation of Almatti Dam on Krishna River (A Case Study) – *Mr. Mahavir Bidasaria, Ferro Concrete Const. (India) Pvt. Ltd*
- Innovative Chris-Cross Grouting Around Vertical Penstocks of Koteshwar HE Project – *Mr. Rajeev Vishnoi, THDC India Ltd.*

Mr. S.P. Kakran delivering his Inaugural Address

A view of the delegates and invitees during the Seminar

- Jet Grouting Techniques for Foundation Improvement – A Case Study of Teesta Stage-V Project, Sikkim – *Mr. N.K. Mathur, NHPC Ltd.*
- Influence of Pre-Grouting during Underground Excavation - Some Case Studies – *Mr. M.M. Madan, GVK Group*
- Importance of Grouting in Hydro Electric Projects with Special Reference to NJHPS (1500 MW) and RHEP (412 MW) – *Mr. R.S. Chauhan, SJVN Ltd.*
- Grouting under Compelling Site Scenario – *Dr. Sunil S. Basarkar, ITD Cementation* India Limited
- Grouting and Permeability in Tunnels – Dr. *Rajbal Singh, Central Soil and Materials Research Station*
- Strengthening of Earthen Dam Foundation – Case Studies – *Mr. Y.P. Singh, WAPCOS Ltd.*

SEMINAR ON SLOPE STABILIZATION CHALLENGES IN INFRASTRUCTURE PROJECTS

20-21 October 2011, New Delhi

The dignitaries on the dais during the Inaugural Session (LtoR) Mr. V.K. Kanjlia, Secretary, CBIP; Mr. R. Jeyaseelan, Former Chairman, CWC; Dr. V.K. Yadav, Addl. Director General, DGBR; Dr. S. Gangopadhyay, Director, CRRI; Dr. H.R. Sharma, Director (D&E), GVK Group and Dr. A.K. Dhawan, Former Director, CSMRS

Dr. H.R. Sharma and other dignitaries lighting the lamp to mark the auspicious beginning of the Seminar

As part of its activities, the Indian National Group of International Society for Rock Mechanics with the support of the Central Board of Irrigation & Power and Indian Chapter of International Geosynthetics Society, organised a Seminar on "Slope Stabilization Challenges in Infrastructure Projects" at CBIP Conference Hall, Malcha Marg, Chanakyapuri, New Delhi on 20 and 21 October 2011.

The Seminar was co-sponsored by TenCate Geosynthetics Asia Sdn. Bhd.

The objective of the proposed Seminar was to provide a forum for design and construction engineers for analyzing the stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and

Dr. V.K. Yadav delivering the Presidential Address

A view of the delegates and invitees during the Seminar

natural slopes in soil and soft rock, methods for analysis of slope stability, strength tests, analysis conditions, and factors of safety, use of geosynthetics for slope stability, etc.

The Seminar was inaugurated by **Dr. S. Gangopadhyay**, Director, Central Road Research Institute on 20th October 2011. The Inaugural Session was presided over **Dr. V.K. Yadav**, VSM, Addl. Director General, Directorate General Border Roads (DGBR) and President, Indian Road Congress.

Dr. H.R. Sharma, Director (Design & Engineering), GVK Technical & Consultancy Services Pvt. Ltd. and President, Indian National Group of ISRM and **Mr. R. Jeyaseelan**, Former Chairman, Central Water Commission also shared their experiences with the participants during the Inaugural Session.

Dr. A.K. Dhawan, Former Director, Central Soil and Materials Research Station and Seminar Coordinator briefly described the objectives of the Seminar during the Inaugural Session.

In total, 70 participants from 34 organizations participated in the Seminar.

Following papers were presented and discussed during the Seminar:

- Major Land Slides and Slope Stability Problems encountered in Hydro Projects - *Mr. R. Jeyaseelan, Former Chairman, Central Water Commission*
- Stability Problems in Natural and Manmade Slopes – *Dr. A.K. Dhawan, Chief Consultant, Fugro Geotech Pvt. Ltd., and Former Director, Central Soil and Materials Research Station*
- Soil Nailing for Slope Stabilization – *Dr. Satyendra Mittal, Associate Professor, Department of Civil Engineering, Indian Institute of Technology Roorkee*
- An Overview of the National Guidelines for Management of Landslides with Particular Reference to Infrastructure Projects – *Dr. Surya Prakash, Associate Professor, Geohazards & Risk Management Division, National Institute of Disaster Management*
- Application of Composite High Strength Geotextile Reinforced Soil Structures with Poor Draining Soil Backfill for Slope Stabilization – *Mr. Kiran Kumar Rumandla, Technical Support Engineer, Tencate Geosynthetics Asia Sdn. Bhd.*
- Stability of Spillway Cut Slope in Dhauliganga HE Project - *Dr. Gopal Dhawan, Executive Director, NHPC Ltd.*
- Numerical Technique for Analyzing Slope Stabilization – *Dr. (Ms.) R. Chitra, Scientist, Central Soil and Materials Research Station*
- Slope Stability: Mitigating Risk in Dragline Dump Slopes – *Mr. Randip Singh, Senior Manager, Opencast Division., Central Mine Planning & Design Institute Limited*
- Gabions for Slope Stability – *Ms. Shabana Khan, Senior Manager, Maccaferri Environmental Solutions Pvt. Ltd.*
- The Importance of the Geological and Geotechnical Characterization for Numerical Modelling - Requirements and Methods - *Dr. Edward Button, Project Manager – Geotechnics, Geoconsult India Pvt. Ltd.*
- Designing of Ultimate Pit and Waste Dump in a Coal Mine using “limit equilibrium” Method of Slope Stability Analysis – *Dr. Sukanya Chakraborti, Assistant Manager (Geology), Tata Consulting Engineers Limited*

The Seminar concluded with the panel discussions on 21 October 2011 under the Chairmanship of **Dr. H.R. Sharma**, Director (Design & Engineering), GVK Technical & Consultancy Services Pvt. Ltd. and President, Indian National Group of ISRM. Dr. Edward Button, Project Manager – Geotechnics, Geoconsult India Pvt. Ltd. also participated in the Panel Discussions as panelist. During the session, participants discussed the problem being faced in the professional engagements and provided their feedback about the Seminar.

ISRM NEWS

Dr. Manoj Verman designated as the President of ISRM Commission on "Hard Rock Excavation"

ISRM has set up a new Commission on "Hard Rock Excavation" and Dr. Manoj Verman, Director (Tunnelling and Geotechnical)., has been designated its President.

The new Commission will run for four years (2011 to 2015) starting in October 2011 in Beijing during the ISRM Congress. The product of the Commission will be a report, titled "Lessons Learnt from Hard Rock Excavation Projects". This would include:

- Compiling case histories of hard rock excavation projects, especially those from the Himalayas, the Alps and the Andes
- Documenting problems faced during hard rock excavation, particularly in difficult ground conditions
- Recommendations for avoiding the problems faced
- Producing a report highlighting the lessons learnt from hard rock excavation projects

ISRM Young Members Presidential Group

One of the modernisation initiatives by ISRM is the formation of a Group of young ISRM members to be called the ISRM Young Members Presidential Group, initially to be a Group of 7; with one young member from each of the six ISRM regions and one nominated by the ISRM President, of less than 35 years old, and preferably be graduate students.

The purpose of the Group and the interaction with the Young Members is to discuss:

- issues the young member has concerning the Society;
- initiatives for making the Society more responsive to the needs of its young members;
- suggestions for promoting the Society to young practitioners;
- initiatives for addressing better or differently the objectives and purposes of the Society; and
- initiatives for increasing the overall membership of the Society.

These young members will have the opportunity and privilege of 'meeting' directly with the President. Therefore their opinions and thoughts will be channeled directly to the ISRM President and hence the Board.

Indian National Group has nominated Dr. V.B. Maji, Assistant Professor, Department of Civil Engineering, IIT Madras, for the ISRM Young Members Presidential Group.

ISRM Council Meeting, Beijing, 17 October 2011

The ISRM held its Council meeting in Beijing, China, in conjunction with 12th International Congress on Rock Mechanics. 45 of the 48 National Groups were either present or represented. The Council was also attended by Past Presidents Prof. Ted Brown, Prof. John Franklin and Prof. Sunshuke Sakurai, the chairmen of the ISRM Commissions, representatives of the IAEG, the IGS and the ITA and the candidates to the election for Vice President 2011-2015.

Celebration of the 50th Anniversary of the ISRM

The ISRM was founded in 1962 by Prof. Leopold Müller in Salzburg and entered its 50th year of existence. A logo for this celebration was chosen in a competition open to young ISRM members, won by Dr Ludger Suarez Burgoa from Bolivia. Another competition took place to select the best slide show on "The Future Directions of Engineering Rock Mechanics", which was won by Dr Ricardo Resende from Portugal. A book about the 50 years of the ISRM history is being prepared and will be launched in Stockholm next May. Other initiatives are being planned, which will be announced to the members.

Membership of the ISRM

The ISRM has now 6514 individual members and 123 corporate members, belonging to 48 National Groups. This represents an increase of 3.5% in the number of individual members since the last year and the highest number of individual members ever. One half of the members come from Europe, and one quarter from Asia. China is currently the largest National Group of the ISRM.

ISRM Digital Library

The ISRM digital library was launched last year in New Delhi and contains the papers and the keynote lectures published in the ISRM Congresses and sponsored Symposia, thus giving them a greater visibility and making them available to all the rock mechanics professionals. The ISRM digital library is part of OnePetro, a large online library managed by the Society of Petroleum Engineers. ISRM individual members registered on the ISRM website are allowed to download, at no cost, up to 100 papers per year from the ISRM conferences, and ISRM corporate members can download 250 papers. To register on the OnePetro website (www.onepetro.org) as an ISRM member, the only necessary information is the username and the password used to access the members' area of the ISRM website. Nonmembers can purchase the ISRM papers online. The ISRM digital library has now 4,000 papers published in 21 conferences, and these numbers will to continue to increase.

The Board of the ISRM for 2011-2015

Following the election of Prof. Xia-Ting Feng from China for ISRM President 2011-2015 two years ago, the Council elected in Beijing the six Regional Vice Presidents:

- Mr Jacques Lucas (South Africa) for Africa,
- Dr Yingxin Zhou (Singapore) for Asia,
- Dr David Beck (Australia) for Australasia,
- Prof. Frederic Pellet (France) for Europe,
- Dr John Tinucci (USA) for North America and
- Dr. Antonio Samaniego (Peru) for South America.

The newly elected Board met immediately after the Council to elect Dr Antonio Samaniego as the 1st Vice President, and to appoint as Vice Presidents at Large

- Prof. Yuzo Onishi from Japan and
- Prof. Ivan Vrkljan from Croatia.

Dr Luís Lamas was reappointed as Secretary General.

Rocha Medal 2012 awarded to Dr Maria Teresa Zandarin from Argentina

The ISRM Board decided to award the Rocha Medal 2012 to Dr Maria Teresa Zandarin from Argentina for the thesis "Thermo-hydro-mechanical Analysis of Joints. A Theoretical and Experimental Study" presented at the Polytechnic University of Catalonia, Spain. She will receive the award at the 2012 ISRM International Symposium in Stockholm.

Two runner-up certificates were also awarded to Dr Bryan Philip Watson from South Africa for the thesis "Rock Behaviour of the Bushveld Merensky Reef and the Design of Crush Pillars" presented to the University of the Witwatersrand, South Africa, and to Dr Joshua Taron from the USA for the thesis "Geophysical and Geochemical Analyses of Flow and Deformation in Fractured Rock" presented to the Pennsylvania State University, USA.

12th ISRM International Congress on Rock Mechanics "Harmonizing Rock Mechanics and the Environment", 18 - 21 October 2011, Beijing, China

The 12th ISRM International Congress on Rock Mechanics has been successfully held during 18- 21 October 2011 in Beijing, China.

The 12th ISRM International Congress on Rock Mechanics (ISRM 2011), one of the top events of ISRM, was opened on October 18 at China National Convention Center, Beijing. More than one thousand of experts and scholars in rock mechanics field from about 50 countries and regions participated in the Congress.

Immediately after the opening ceremony, the past ISRM President, Professor Ted Brown delivered a keynote lecture on "50 Years of the ISRM and Associated Progress in Rock Mechanics" and reviewed the history of ISRM and the international rock mechanics field in the past 50 years. Professor J.A. Hudson presented a keynote on "The Next 50 Years of the ISRM and Anticipated Future Progress in Rock Mechanics". He predicted the future 50 years of ISRM and the prospects of rock mechanics in the next 50 years.

Following that, Professor J.A. Hudson presented the ISRM Müller Award to Professor N. Barton. Professor N. Barton presented the Müller lecture.

Sufficient academic exchanges promoting rock mechanics to a new 50-year. The success of the Congress is also reflected in the sufficient and fruitful academic exchanges. 15 keynote papers were presented, including three keynotes for the 50th anniversary of ISRM, the Müller lecture, the Rocha Medal lecture, and ten invited keynotes. 227 oral presentations in four parallel sessions and 140 post presentations in the foyer were delivered.

The ISRM 50th anniversary celebration was held in the evening of 20th October. Professor N. Grossmann introduced the foundation and development history of ISRM, the hosting time, country and city of previous ISRM congresses and the past board members.

The closing ceremony was hosted by the ISRM Secretary-General, Dr. L. Lamas, on 21st October. The newly-elected board members took over from the current board members. Professor Xiating Feng received the scepter and ribbon from Professor J.A. Hudson and officially became the ISRM president. He then delivered his inaugural speech and expressed his determination to expand the ISRM, to improve the benefits to national groups and members and to improve effectiveness of the ISRM services.

Election of Dr. Manoj Verman as ISRM Vice President at Large

Dr. Manoj Verman from India has been elected as 3rd ISRM Vice President at Large for the period 2011-15.

The newly elected ISRM Board, under the President ship of Prof. Xia-Ting Feng, decided that in order to encourage Asian National Groups to be more involved in ISRM activities, the Board may have another ISRM Vice President at Large for the term 2011-2015 from Asia if it has a suitable candidate, and this invited nominations from Indian National Group, besides from Iran and Korea. Indian National Group submitted the nomination of Dr. Manoj Verman, as approved by the Governing Council of the Society in its meeting held on 15 November 2011.

Nomination for ISRM Rocha Medal 2013

Indian National Group has nominated Dr. Prabhat Kumar Mandal, Principal Scientist, Central Institute of Mining and Fuel Research, for Rocha Medal 2013, for his thesis on "Development of a Methodology for Underground Extraction of Contiguous and Thick Contiguous Seams/ Sections under Weak and Laminated Parting".

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.

The updated manual contains the following 19 chapters:

1. Rock Mechanics – Need and Scope
2. Rock Mechanics- Basic Concepts
3. Laboratory Assessment of Rock
4. Site Characterization – Quantitative Description of Discontinuities in Rock Masses
5. Classification of Rocks Intact and Mass
6. Geophysical Investigations
7. Drilling for Geological Investigations
8. Laboratory Tests for Design
9. State of Stress in Rock Mass
10. Field Tests for Design
11. Devices and Instruments to Measure Movements and Pressures
12. Interpretation of Test Data
13. Rock Loads and Tunnel Supports
14. Design of Tunnels with Rock Support Interaction Analysis
15. Design of Tunnel Lining
16. Numerical Methods in Rock Engineering
17. Foundation Treatment of Dams
18. Rock Blasting for Underground and Open Excavations
19. Shotcreting, including Some Case Histories

ISRM SPONSORED FORTHCOMING EVENTS

- 28-30 May 2012, Stockholm, Sweden - *EUROCK'2012 – Rock Engineering and Technology*: the 2012 ISRM International Symposium.
- 8-10 August 2012, San Jose, Costa Rica - *II ISSER – South American Symposium on Rock Excavations*: an ISRM Regional Symposium.
- 15-19 October 2012, Seoul, Korea - *the 7th Asian Rock Mechanics Symposium*: an ISRM Regional Symposium.
- 21-26 September 2013, Wroclaw, Poland - *EUROCK' 2013 - Application of Rock Mechanics to Civil and Mining Engineering*: an ISRM Regional Symposium.
- 20-22 May 2013, Brisbane, Australia - *Effective and Sustainable Hydraulic Fracturing*: an ISRM Specialized Conference
- 20-22 August 2013, Sendai, Japan, *The 6th International Symposium on in situ Rock Stress*: an ISRM Specialized Conference.
- 26-28 May 2014, Vigo, Spain - *EUROCK' 2014 - Rock Engineering and Rock Mechanics: Structures in and on Rock Masses*: an ISRM Regional Symposium.
- 29 April-6 May 2015, Montréal, Canada - *Innovations in Applied and Theoretical Rock Mechanics*: the 13th ISRM International Congress on Rock Mechanics.

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 **c**ontribute 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.

$$E=mc2 \quad (1)$$

- **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.
  - o Jones B., Brown, J., and Smith J. 2005**,** The title of the book. 1st edition, Publisher.
  - o Jones B., Brown, J., and Smith J. 2005 The title of the conference paper*. Proc Conference title* 6: 9-17.
  - o 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**

ABOUT ISRM

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 Society has 5,000 members and 46 National Groups.

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;
- 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 main activities carried out by the Society in order to achieve its objectives are:

- to hold International Congresses at intervals of four years;
- to sponsor International and Regional Symposia, organised by the National Groups the Society;
- to publish a News Journal to provide information about technology related to Rock Mechanics and up-to-date news on activities being carried out in the Rock Mechanics community;
- to operate Commissions for studying scientific and technical matters of concern to the Society;
- to award the Rocha medal for an outstanding doctoral thesis, every year, and the Müller award in recognition of distinguished contributions to the profession of Rock Mechanics and Rock Engineering, once every four years;
- to cooperate with other international scientific associations.

The Society is ruled by a **Council**, consisting of representatives of the **National Groups**, the **Board** and the Past Presidents. The current President is Prof. John A. Hudson, from United Kingdom.

The **ISRM Secretariat** has been headquartered in Lisbon, Portugal, at the *Laboratório Nacional de Engenharia Civil* - LNEC since 1966, date of the first ISRM Congress, when Prof. Manuel Rocha was elected as President of the Society.

BENEFITS TO MEMBERS

The current benefits given to ISRM members are:

- ***Individual and corresponding members***
  - 1 copy of the ISRM News Journal
  - ISRM Newsletter
  - Access to the members area in the website (download of Suggested Methods and Reports, Rock Mechanics lectures, videos, slide collection, etc.)
  - Right to participate in the ISRM Commissions
  - Registration with a 20% discount in the ISRM Congress, International and Regional Symposia and Specialised Conferences
  - Personal subscription to the International Journal of Rock Mechanics and Mining Sciences at a discounted price (see details).
  - Personal subscription to the Journal Rock Mechanics and Rock Engineering at a discounted price.
  - Free download of up to 100 papers per year from the ISRM Digital Library at OnePetro: www.onepetro.org
- ***Corporate members***
  - Listed in the ISRM website, with a link to the company’s website
  - Listed in the ISRM News Journal
  - Access to the members area in the ISRM website
  - ISRM Newsletter
  - 1 copy of the ISRM News Journal
  - 1 registration with a 20% discount as ISRM member in the ISRM Congress, International and Regional Symposia and Specialised Conferences
  - Free download of up to 250 papers per year from the ISRM Digital Library at OnePetro: www.onepetro.org

INDIAN NATIONAL GROUP OF ISRM

Introduction

The study of rock mechanics has assumed considerable importance because of its wide application in civil engineering, more predominantly in water resources, mining engineering and underground structures. For the execution of multipurpose water resources projects located in complicated geological settings, the significance of rock mechanics in the design and construction was realised in late 1950s. Despite tremendous alround advancement in technology, a full understanding of natural forces and phenomena eludes the design engineer. Liberalisation of economy has facilitated planning and execution of many exciting and complicated projects. These projects require application of modern principles of rock mechanics, which warrants deliberations and collaboration to facilitate flow of appropriate technology to enable successful implementation of such projects under a time-bound programme in a cost-effective manner, conforming to environmental requirements.

The Indian National Group of ISRM - ISRM (India) is involved in dissemination of information regarding 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 organisations.

ISRM (India) represents International Society for Rock Mechanics, founded in Salzburg in 1962, as its Indian National Group.

Objectives

- to encourage collaboration and exchange of ideas and information between rock mechanics practitioners in the country
- to encourage teaching, research and advancement of knowledge in the rock mechanics
- 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
- to hold events periodically on rock mechanics and rock engineering themes of general interest to the majority of the membership
- to cooperate with international bodies whose aims are complementary to those of the society
- to encourage the preparation of internationally recognized nomenclature, codes of practice, standard tests and procedures
- to encourage collaboration with and support of international programme in the field of Rock Mechanics including cooperation with other organizations in the activities of common interest
- to act as a coordinating National Body of International Society of Rock Mechanics, comprising of members in the country concerned with Rock Mechanics

Membership Fee

- Individual Membership for one calendar year: Rs. 600.00
- Individual Membership for 10 calendar years: Rs. 6,000.00
- Individual Membership for 20 calendar years: Rs. 10,000.00
- Institutional Membership for 01 calendar year: Rs. 10,000.00
- Institutional Membership for 02 calendar years Rs. 18,000.00
- Institutional Membership for 03 calendar years Rs. 25,000.00
- Institutional Membership for 05 calendar years Rs. 40,000.00

For membership and other details, please contact:

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