African Journal of Science and Technology (AJST)
Science and Engineering Series Vol. 4, No. 2, pp. 42-50

SELECTION AND TESTING OF BALLAST STONES FOR UNDERGROUND RAILWAY TRACKS

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ABSTRACT: *Ballast is broken pieces of hard rocks such as sandstones, schist, etc. approximately 25-60 mm size, over which the railway tracks are laid. The function of the ballast is to transfer the applied load over a large surface, provide adequate elasticity, prevent creep and hold the sleepers in position. Also under wet conditions, it would permit free drainage and allow free grade to be obtained. It is reported that a large proportion of serious accidents occur through derailments of carriages. Many such accidents may be due to the poor quality of ballast stones. The paper discusses the essential properties of ballast stones and methodologies for testing these properties.*

INTRODUCTION

The introduction of track locomotives for carrying men and materials cannot be over emphasised in the present industrial world. A general increase in the travelling distance between the shaft and the face has resulted in establishment of high speed underground rail system in many mines. In mines where the standard of track is good 40 kmph have been achieved. This reduction in travelling time has significantly increased the productive period spent by miners at the face during each shift.

The motion of a railway vehicle travelling along a mine roadway is dependent on such things as the suspension of the vehicle, its operating speed, and the quality of the track it is running on.

The essential requirements of the track, apart from sleepers, is ballast stones over which the tracks are laid. This must be capable of carrying the load imposed upon it with safety and security over a long period of time, with no risk of derailment and a minimum cost of repair and maintenance.

It has been observed that a large proportion of serious accidents occur through derailment of carriages. Many such accidents are primarily due to the poor quality of ballast stones.

The paper discusses the pertinent properties of ballast stones and the procedures for testing these properties.

SOME PERTINENT PROPERTIES OF BALLAST STONES

General Requirements

The purpose of ballast stones is to distribute the intense bearing pressure of the wheel on the trail over a sufficient area of floor so that the safe bearing pressure of the floor is not exceeded and hold the sleepers in place . Also, under wet conditions these stones should permit free drainage of water.

The ballast used must not crumble or disintegrate due to wet conditions, must bind well together but remain porous and elastic throughout and must be hard and durable and remain unweathered. The ballast stone should not contain inorganic or organic residues and its contamination with ground soil during production and stacking should be minimised. As far as possible, ballast should be of angular shape and should consist of a mixture of sizes as given in Table 1.

Table 1 Grading of ballast stone

Type	Mesh	% Retention
	60	Nil
	50	Not exceeding 10%
50 mm	20	Not exceeding 25%
	12	Not exceeding 100%
	40	Not exceeding 10%
25 mm	25	Not exceeding 25%
	12	Not exceeding 100%

Mechanical Properties of Ballast

The principal mechanical properties of ballast stones are:

(a) Abrasiveness
(b) Slake durability
(c) Fracture toughness

Also, recent works have shown that some physical properties of ballast such as compressive strength, porosity, elasticity, thermal expansion, bond characteristics, volume change on wetting and drying and shape (angularity and flakiness) have profound influence on the suitability of stone as ballast. The following sections briefly describe methods of determining principal mechanical properties of ballast stones some by direct methods and others by indirect methods which are economical and easy to perform.

Abrasion test

Abrasion test measures the resistance of rocks to wear. The abrasiveness of rock is dependent on the type and quality of various mineral constituents of the rock and bond strength that exists between the mineral grains. The method described below covers procedure for testing aggregate for resistance to abrasion using the Los Angeles testing machine shown in Figure 1.

The test samples and the abrasive charge (which are dependent upon the aggregate size and grading) are placed in the Los Angeles testing machine (shown in Figure 1) and the cylinder is rotated at a speed of 30-33 rev/min. The number of revolutions can be increased to 500 per minute for aggregate smaller than 38 mm. The machine is so driven and counterbalanced as to maintain a substantially uniform peripheral speed. After the prescribed number of revolutions, the material from the cylinder is discharged and sieved on 1.7 mm. The material greater than 1.7 mm is washed and weighed.

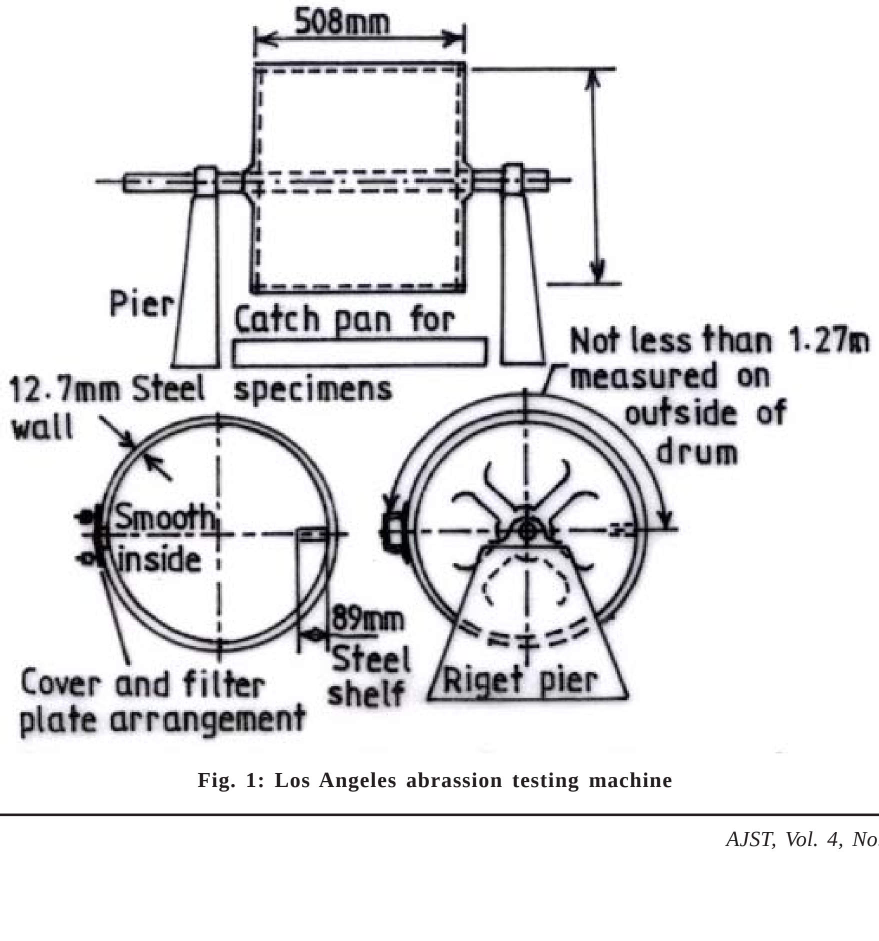

Fig. 1: Los Angeles abrassion testing machine

The difference between the original weight and the final weight of the test sample is expressed as a percentage of the original weight of the test sample. This gives the aggregate abrasion (wear) of the test material.

In the Los Angeles abrasion test, the stone should not have a percentage loss of more than 40% after 500 revolutions to qualify for use as ballast.

Slake durability test

Since rocks containing high clay content are prone to swelling and disintegration when exposed to short term weathering process of wetting and drying, special tests are necessary to predict their mechanical performance and for the purpose of comparing one rock to another.

The test is intended to assess the resistance offered by a rock sample to weakening and disintegration when subjected to two standard cycles of drying and wetting.

Methods of determining values for the above properties are well described in text books on rock mechanics.

Fracture toughness testing

Fracture toughness is a property of a rock expressing its resistance to catastrophic crack propagation or from the energy point of view, it is the fracture surface energy required to create unit new crack surface.

The fracture toughness of rock can be interpreted by the three fracture parameters: the critical stress, intensity factor, critical I-integrate and specific work of fracture.

Figure 2 shows the Chevron fracture testing apparatus for determining fracture toughness of rock. The specimen configuration is illustrated with loading and geometry notations. The international society for rock mechanics (ISRM) suggested dimensions of test specimen given in Table 2.

Notations:

A = Ligment area
D = Specimen diameter
l = Loading span 3.33 D
a = Crack length
a_o = Chevron tip distance from specimen surface, 0.15 D
h = Depth of cut in notch flank
B = Crack front width
t = Notch width
L = Specimen length
P = Applied load
2θ = Chevron angle, 90º

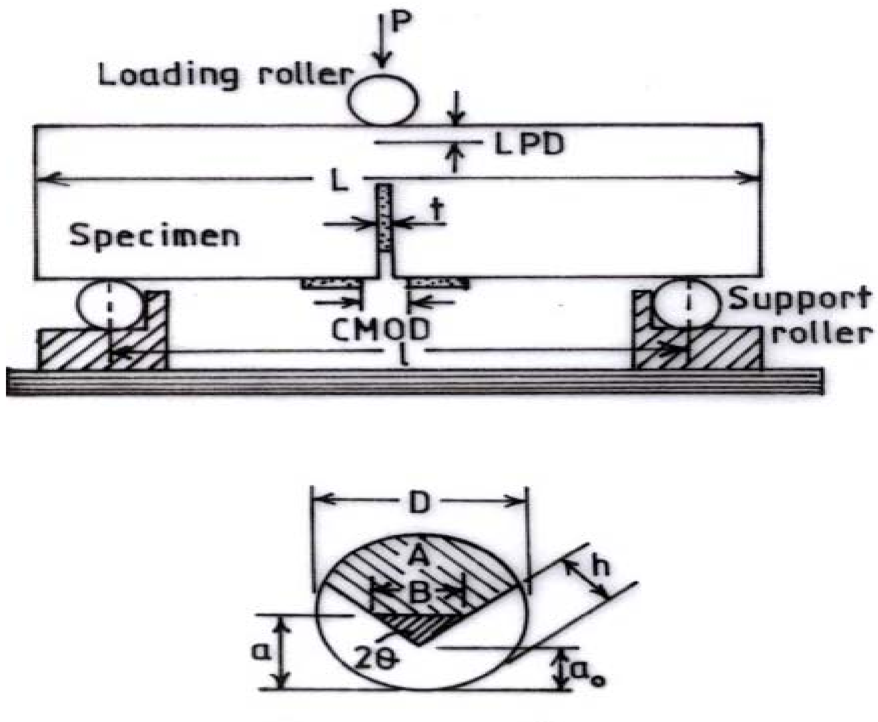

Fig. 2: Chevron fracture testing apparatus (after ISRM 1988)

Table 2. ISRM suggested dimensions for Chevron Notch in Bending (CB) specimens

Geometry Parameter	Value	Tolerance
Specimen diameter	D	>10 x grain
Specimen length	4D	>3.5D
Half chevron angle, Q	45°	± 1.0°
Chevron V tip position, a_o	0.15D	± 1.0D
Loading span, I	3.33D	± 0.02D
Notch width, t	≤ the greatest of 0.03D and 1 mm	-

The chevron notch in bending (CB) causes crack propagation to start at the tip of the V and proceed in the Chevron notch plane in a stable way until applied load reaches its maximum value. This value is used to estimate the fracture toughness.

For a round bar with a single straight-through crack in three-Point, bending as shown in Figure 2, the stress intensity factor K_1 is given by Bush (1976) as follows:

$$K^s{}_1 = \frac{P}{D^{3/2}} x Y^s{}_k \qquad (1)$$

where, $$Y^s{}_k = \frac{1}{8D}\sqrt{\frac{1}{\sqrt{\alpha-\alpha^2}}\frac{\partial(Y^s\lambda}{\partial\alpha}} \qquad (2)$$

α = a/D, the dimensionless crack length
D = the diameter of the round bar
∂ = the loading span

lthough the above methods of ISRM are the recent innovations for determining fracture toughness, they are inherent difficulties in preparing specimens. they require sophisticated systems of loading and recording, crack length measurement techniques and reduction process. In view of this, the Brazilian disk (in diametral compression) still remains a popular method for this purpose.

Figure 3 shows schematically the specimen configuration of a rock specimen, containing a single edge crack subjected to diametral compression.

The test was developed by Szendi-Horvath (1980) to determine fracture toughness of brittle materials. The crack in this case is initiated by the transverse tensile stress resulting from the diametral compression. Since the tensil stress is always at its maximum in the centre of the disk (Hondros 1959) the crack initiation takes place from the crack tip in or near the centre of the disk.

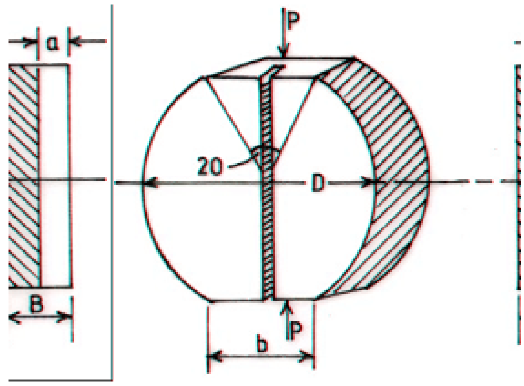

Fig. 3: Brazilian disk-type specimens in diametral compression having grooved disk with an edge crack

The central part of the crack propagates outwards in the diametral direction until it is stopped by the compressive stress near the loading surface. The specimen pre-cracking is not required if a saw-cut notch is used (instead of a sharp crack).

The maximum stress, which is at the centre of the crack of the disk, is given by:

$$\sigma_\theta = \frac{0.6366\,(\sin 2\theta\text{-}\theta)P}{BD\sin\theta} \qquad (3)$$

Where

P = Diametral compressive load
D = Disk diameter
B = Disk thickness
θ = Crack orientation angle with respect to the applied load

The central part of the disk can be considered as a semi-infinite plate subjected to a uniform tensile stress σθ as long as the ratio of the crack depth to specimen, a/B is relatively large.

For a plate of finite size, the stress intensity factor (for mode I) is given by:

$$K1 = 1.12\sigma(\pi a)^{1/2} \qquad (4)$$

Where the correction factor is 1.12 (which is approximately polynomial of fourth order) by combining equation (3) with (4).

$$K_I = \frac{1.264(\sin 2\theta - \theta)P\sqrt{a}}{BD\sin\theta} \qquad (5)$$

For a line load, i.e. $\theta = 0°$, the limit of the above Expression when θ approaches 0° becomes as follows:

$$K_I = \frac{1.264P\sqrt{a}}{BD} \qquad (6)$$

The fracture toughness can be obtained by substituting the critical load P in the corresponding expression K_I given above.

NEED FOR INTERRELATING ROCK HARDNESS TO FRACTURE TOUGHNESS

Due to the complexity of rock structures it has been observed that it is more difficult to obtain fracture toughness than to determine 'hardness index' and some mechanical properties of rocks. Furthermore, different testing procedures for determining fracture toughness may yield substantial differences in the results. Also, most rock testing laboratories (including commercial ones) are well equipped to carry out hardness index tests and mechanical properties than determining fracture toughness. Index tests are quick, easy to perform and less expensive.

The following sections discuss the relationship between the hardness index values obtained by various methods and the fracture toughness.

The National Coal Board (NCB) Cone Indenter

The National Coal Board (NCB) cone indenter is illustrated diagrammatically in Figure 4. The instrument consists of a portal steel spring leaf fixed along its horizontal axis. In the middle of one side of the frame, a dial gauge is mounted in such a way that its probe is in contact with the spring leaf that can be easily detected and accurately measured. On the opposite side of the frame, a micrometer is mounted with a tungsten carbide cone inserted in its spindle. It measures the depth of penetration of the cone plus the deflection of the spring leaf. This portable instrument is able to give a measure of rock hardness (denoted hereby H_{CI}), derived from the force (the spring deflection) necessary to cause a certain amount of penetration.

The following equation is used to calculate the cone indenter index:

$$H_{CI} = D/P \qquad (7)$$

Where, H_{CI} = NCB cone indenter Hardness index
D = Deflection of spring leaf (mm)
P = Penetration of cone (mm)

Fig.5 shows the relationship of fracture toughness to NCB cone indenter hardness index.

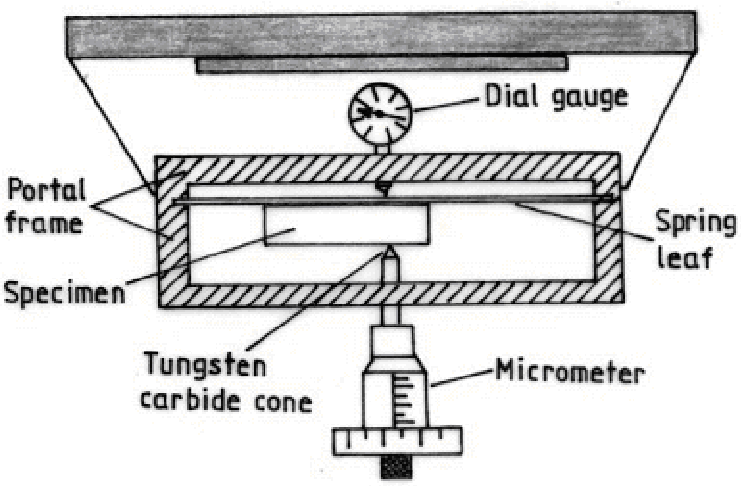

Fig. 4 Diagrammatical illustration of the NCB cone indenter

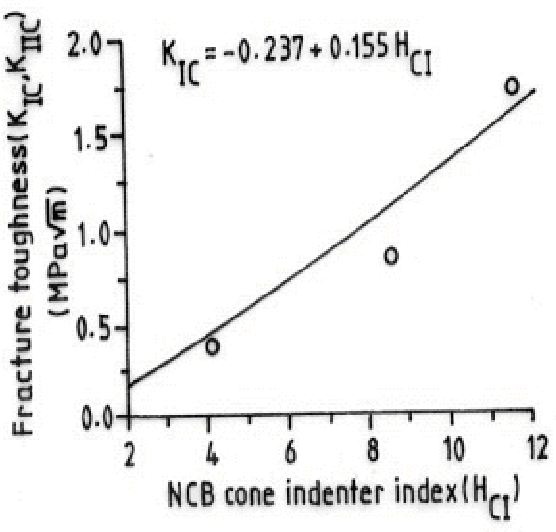

Fig.5 Relationship of fracture toughness to NCB cone indenter (after whittaker, B.N., 1987)

The Shore Scleroscope Hardness index

The shore Scleroscope was invented by Albert F. Shore in the USA and has recently been used to determine the hardness of rock. This method of testing hardness of rock has no limitation on the shape of size of the rock.

The shore scleroscope is illustrated in Figure 6.

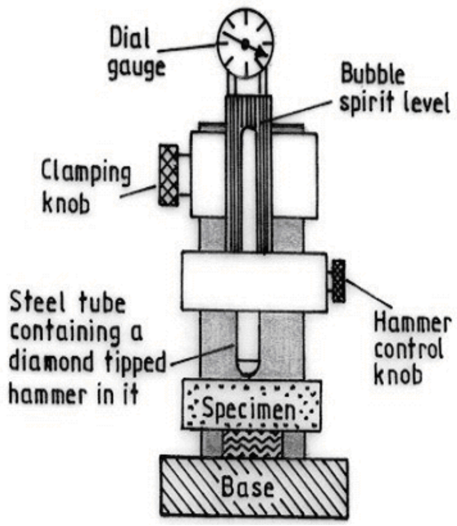

Fig.6 Diagrammatical illustration of shore scleroscope

The instrument consists of a vertical steel tube containing a diamond tipped hammer. The hammer is dropped from a predetermined height on to the surface of the rock specimen and the rebound height which varies depending upon the hardness of the rock is measured. The relationship between the fracture toughness and the shore scleroscope hardness index is shown in Fig. 7.

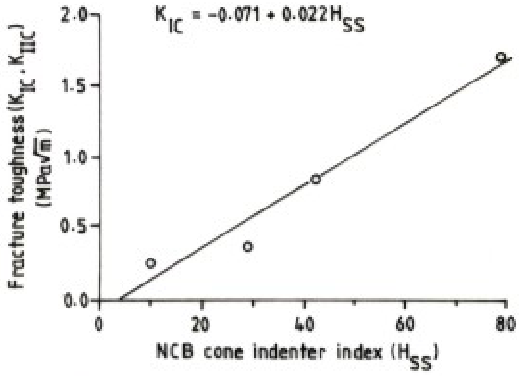

Fig. 7 Relationship of fracture toughness to shore scleroscope hardness index (After Whittaker, B.N., 1992)

Some Physico-mechanical properties and their correlations with fracture toughness

Methods for determining values for properties such as uni-axial compressive and tensile strength, point-load strength, Young's modulus and acoustic wave velocity are well described in text books on rock mechanics. Therefore, only the graphs showing the relationships and the correlation coefficients have been shown in Figures 8,9,10,11 ,12 and 13.

The statistical analyses of results have shown that there are close relationships between fracture toughness and hardness index and some physico-mechanical properties of rock, fairly close linear relationships exist between fracture toughness and unixial tensile strength whilst favorably close relationships exist with uniaxial compressive strength, point-load strength, flexural strength and Young's modulus.

These correlations of fracture toughness with hardness and physico-mecanical properties may be used as a basis for indirectly evaluating fracture toughness from hardness index or from any of the physico-mechanical properties described in this section.

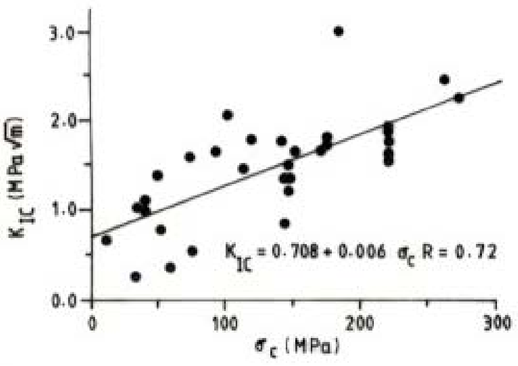

Fig. 8: Correlation of mode 1 fracture toughness with uniaxial compressive strength

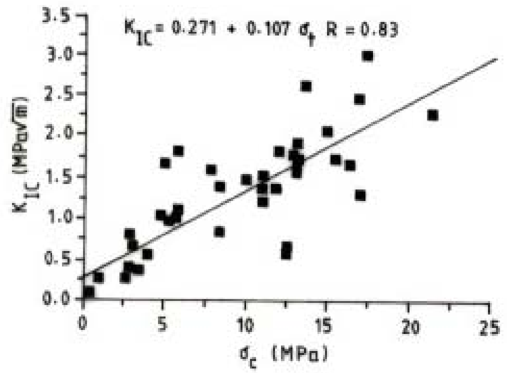

Fig. 9: Correlation of mode 1 fracture toughness with uniaxial tensile strength

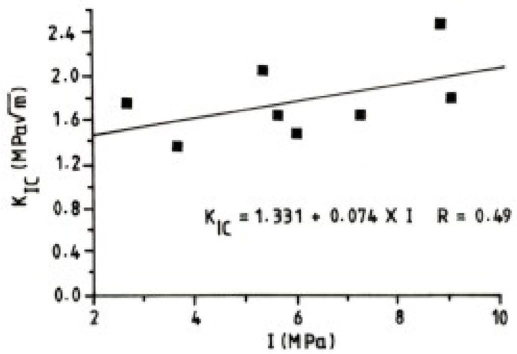

Fig. 10: Correlation of mode 1 fracture toughness with point-load strength index

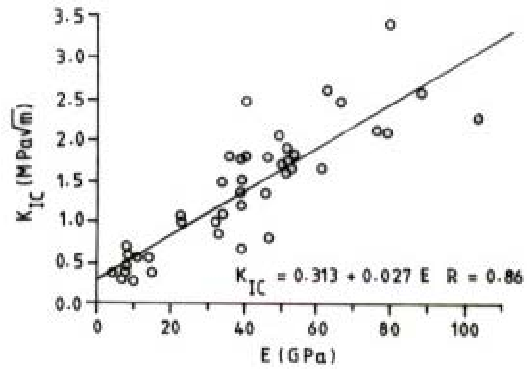

Fig. 11: Correlation of mode 1 fracture toughness with Young's modulus

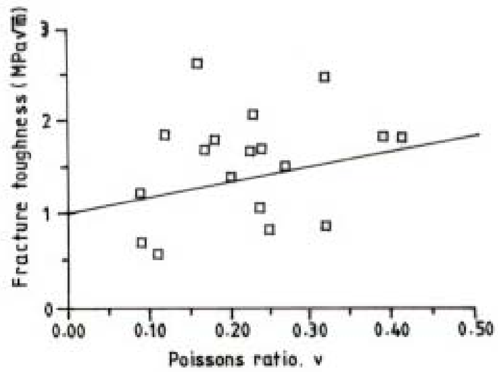

Fig. 12: Plot of mode 1 fracture toughness against Poisson's ratio

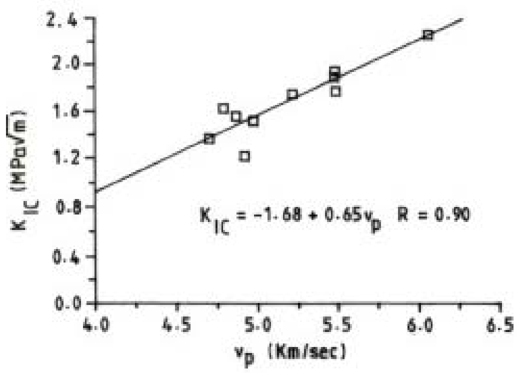

Fig. 13: Correlation of mode 1 fracture toughness with velocity of acoustic wave

The advantages of these tests are apparent and cannot be overemphasised since these are much less expensive and time consuming and above all, easy to perform than the fracture parameter tests.

Table 3 below shows the emperical relationships between rock fracture toughness and the various physico-mechanical properties.

Table 3 Emperical relationships between rock fracture toughness and some Physico-mechanical properties

Physico-mecha-nical Tests	Emperical relationships	Correlation Coefficients ®	Reference
NCB Cone Ludenter Number	KIC=-0.24+0.16HIC	-	Singh and Sun, 1989
Shore Scleroscope Number	KIC=-0.72+0.02Hss	-	Gunsallus and Kulhawy, 1984
Uniaxial Tensile Strength (Mpa)	KIC=-0.27+0.11st	0.83	Schmidt, 1977
Uniaxial compressive Strength (Mpa)	KIC=-0.71+0.006sc	0.72	Fong and Nelson, 1986
Point-load strength (Mpa)	KIC=-1.33+0.07I	0.49	Gunsallus and Kulhawy, 1984
Young's modules (Gpa)	KIC=-0.313+0.03E	0.86	Gunsallus and Kulhawy, 1984
Velocity of acoustic wave (Km/Sec)	KIC=-1.68+0.65Vp	0.90	Huang and Wang 1985

LAYING OF BALLAST STONES

Figure 14 shows ballast stone arrangements for safe and efficient railway carriages. The ballast rock must not crumble or disintegrate in use and must remain porous and retain its elasticity.

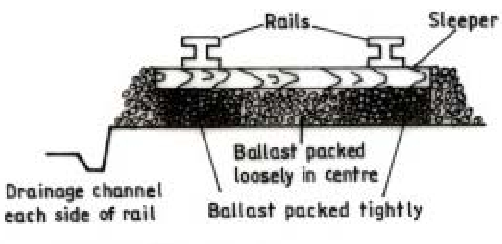

Fig. 14: Ballast stones arrangements for railway tracks (after NCB Handbook)

The initial layer of ballast should be of coarser material, 37-50 mm square or 50 mm round mesh while finer chippings should be used for packing under the sleepers. In wet conditions an extra initial layer of large materials, 100-200 mm may be necessary for better drainage. Depending upon the amount of water expected and the type of floor a gradient between 1 in 45 to 1 in 20 should be maintained.

Ballast should be packed tightly for a distance of 150 mm from both ends of the sleepers to prevent sideways creep. If this is not carried out properly the sleepers may rock and may break in the centre. Above all, careful routine examination is necessary for both smooth and safe running of locomotives.

CONCLUDING REMARKS

The work presented in this paper is aimed at improving the use of conventional type of ballast stones and track laying. These are important considerations for the safe operations of railway transport not only for underground railway operations but also for surface rail transport operations.

In this study, it was observed that there are close relationships between rock fracture toughness and hardness index and physico-mechanical properties. This can be regressed by linear equation forms with a high degree of correlation. These correlations with hardness and physico-mechanical properties may be used as a basis for indirectly evaluating toughness from any one of the physico-mechanical properties. Such a method of study and prediction of the fracture behaviour of rocks can lead to an improved understanding of track haulage and thus facilitate the selection of appropriate ballast stone.

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