# Rock Toughness Index For Excavatability Assessment.

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**Summary** The uniaxial compressive strength of a rock substance has traditionally been used as an indicator of the relative difficulty of cutting it, with a roadheader or tunnel boring machine. Other measures of strength which are used are the tensile strength and fracture toughness, which in principle involve a mode of rock failure which is more relevant to the field cutting situation than unconfined crushing. A new measure of rock toughness, "Rock Toughness Index", is defined here, and its correlations with the other 3 measures of rock strength are discussed. The new Index could in future help in the estimation of the cutting rates of roadheaders and tunnel boring machines, and in giving advance warning of the possible danger of rockbursting in deep highly stressed mines and tunnels.

## 1. Introduction

Rock, when considered as an engineering material, often has a degree of natural variability higher than that of most other engineering materials. A logical deduction from this fact would be that the volume of testwork carried out to characterise rock would usually be greater than that considered necessary to prove the properties of metals and artificial rocks (concrete), but this is seldom the case. Design of excavations in and excavation systems for rock has to proceed with an inherent high degree of uncertainty, tempered by the judgment and experience of geotechnical engineers.
An important part of the business of geotechnical engineers is to provide numerical values of rock properties which may accurately characterise the ease or difficulty of excavating rock. In many instances this excavatability of rock has to be deduced, from known or assumed relationships between some measurable rock property and past field experience of rock excavation. In other instances, the correlated rock property is not known, because the volume of available samples and/or the testing budget were insufficient, and a double prediction process is engaged in. From one index test, another rock property is predicted, from which the excavatability is predicted. Uncertainty and inaccuracy of predictions are inherent in this process, as "average" relationships and lines-of-best-fit are used to make each prediction, and a high degree of scatter of the values of physical properties may be expected in rock.

The rock property most widely used and understood in mining and tunnelling is the uniaxial compressive strength. Practical mining and tunnelling engineers make statements such as "the rock had a hardness of 150MPa". Such a statement, though incorrectly mixing units, does reflect a feeling that the resistance of a rock to excavation is a function of something more than its resistance to compressive failure, and the words "hardness", "toughness", and "strength" may be in the back of the mind of the statement's maker.

### 2.1 Point Load Strength Index

Uniaxial compressive strength values are sometimes quoted, with footnotes stating that they were measured with a point load strength tester. The point load strength tester causes tensile failure of a specimen, but this indirect tensile strength is multiplied by a conversion factor, and the converted strength quoted as "uniaxial compressive strength".
How is this conversion factor chosen?
By plotting point load strength index (as *x* values) versus uniaxial compressive strength of the same rock material (plotted as *y* values) and fitting a regression line to the scatter diagram, the slope of the regression line may be taken as therefore implying that the average value of the uniaxial compressive strength is some multiple of the point load strength index. It is not necessarily a causal relationship. The slope found by Broch and Franklin (1972) for a group of 15 rock samples was 24 i.e. U.C.S. $\approx 24I_{s(50)}$. They calculated a correlation coefficient of 0.88, and the ratio U.C.S./$I_{s(50)}$ for their individual rock specimens ranged from 12.4 to 35.7 Despite this wide degree of scatter, this conversion factor of 24 has become enshrined in practice, despite the fact that other authors have reported factors ranging from 5 to 50, for different rock types. Franklin (1985) warned that "*errors of up to 100% are possible in using an arbitrary ratio value to predict compressive strength from point load strength*", and Franklin & Dusseault (1989) warned that "*the prediction of uniaxial strength from point-load strength is unreliable unless confirmed by running both types of test on the same rock type. Point-load strength is best used directly for rock classification rather than as a means of predicting uniaxial compressive strength*."

Analysis of the author's data bank shows that the linear regression through the origin, for all rock types combined has a slope of 16.9 i.e. U.C.S. = $16.9I_{s(50)}$.
The linear regressions for Sandstones and Metamorphics have slopes of 15.3 and 18.5 respectively. ( For each of these 3 cases, the line of best fit was in fact a power curve. )
The ratio of 24 does not seem to apply to the large number of Australian rocks tested by the author.
The use of an "average" strength ratio to manipulate data from a "non-average" rock can lead to inaccurate prediction of uniaxial compressive strength from the point load strength index; the end user of the test data may read it as a measurement rather than as a prediction, and be understandably troubled when the rock actually excavated appears to have markedly different strength from that "measured" by the geotechnical engineer. Some wildly inaccurate predictions of uniaxial compressive strength have been made using the U.C.S. = $24I_{s(50)}$ relationship. The uncritical acceptance of a reported uniaxial compressive strength, without querying how it was derived, can lead an excavation engineer into trouble.

## 2.2 Uniaxial Compressive Strength

Even when uniaxial compressive strength is directly measured, rather than inferred from point-load tests, uniaxial compressive strength test values are, despite their widespread acceptance, an imperfect predictor of rock excavatability. Uncertainty as to the validity of uniaxial compressive strength values, as a guide to rock excavatability, can arise from :
(1) Mode of failure;
(2) Specimen size;
(3) Specimen shape.
To draw the most appropriate deductions from uniaxial compressive strengths an excavation engineer should know all 3 of the above conditions, before knowing what the reported strength really means.
Too often the practical engineer concentrates on the "bottom line" strength number, and ignores the apparently trivial specimen dimensions and mode of failure, to his possible peril.

## 2.3 Modes of Failure

The mode of failure in a laboratory compression testing machine is not necessarily the same as that in the mine or tunnel, under the action of a bit or pick or cutter. In the field cutting or breaking situation the tensile strength is likely to be of great significance; in the laboratory the mode of failure may be by axial cleavage (in which case the tensile strength was in fact the limiting factor), by shear through the rock substance, or by failure along a pre-existing plane of weakness. Results from failures along planes of weakness may give a falsely low estimate of rock mass strength, and should be discounted as predictors of excavatability; however, having paid for the test to be performed, it is understandable that a geotechnical consultant will quote the measured test values in a report to a client.

## 2.4 Size Effect

There is general agreement that there is some form of inverse relationship between specimen strength and specimen volume (i.e. a larger specimen is weaker than a smaller specimen, in terms of applied stress - Not Force - required to break it). There is less agreement as to the actual form of such relationship for any particular rock type e.g. linear, inverse power, logarithmic, exponential , and whether there is a threshold size, above or below which the relationship becomes constant.
The excavating engineer must be wary however, in translating his experience of what a particular uniaxial compressive strength means in terms of excavatability, if his experience was gained, say in interpreting the strength results from 50mm diameter granite cores, when he is now faced with, say the results from 75mm diameter cores of metamorphic rocks. His intuitive deductions may be slightly, but significantly erroneous, because of an unappreciated scale effect.

## 2.5 Shape Effect

The well-known shape effect results in specimens with large Length:Diameter ratios exhibiting lower uniaxial compressive strengths than specimens of the same rock having low Length:Diameter ratios. In some cases the core supplied from field exploration, due to natural jointing, does not allow all the specimens to be prepared having the same standard length; the interpretation of strength values should only proceed after checking the reported specimen dimensions, as the apparent strength variability may be in part a consequence of shape variability.
The standard uniaxial compressive strength specimen shapes recommended by ASTM and ISRM are right cylinders with Length:Diameter ratios of 2.5±0.5, which are designed to allow shear failure to occur, assuming the Mohr-Coulomb failure criterion. It is arguable that the uniaxial compressive strength values derived from "standard" shape specimens, while quite valid for designing stable excavations, and calculating the probability of failure caused by tangential rock stresses, are not necessarily appropriate for the opposing consideration, of calculating the stress required to break rock. In the former case the maximum stresses are assumed to be produced by concentration of natural rock stresses, and are assumed to act parallel to the surfaces of the rock excavation (i.e. in an essentially unconfined mode). In the second case the stresses may be assumed to be applied by artificial

means, in a plane normal to the surface of the rock (i.e. in a partially confined mode). It seems logical then that the same strength characteristics may not apply to both design cases.

### 2.6 "Non-Standard" Compression Testing

This author is conscious of the tension which may develop between attempting to run a standard testing laboratory, performing tests under national or international standard conditions, and attempting to provide test values which are most relevant to industry users (where the needs of the industry users may not have been properly appreciated or taken into account by the drafters of the standards).

Industry experts may not feel able to rely upon the standardised rock testing procedures, and develop their own "non-standard" procedures. For example, Dr. Karlheinz Gehring, the head of the geotechnical department of Voest-Alpine performs all his laboratory rock testing on cores prepared with a Length:Diameter ratio of 1.0, 50mm diameter by 50mm long. This maximises the measured strength, virtually forcing the failure mode to be axial cleavage or pillar-splitting, by making shear failure through the rock substance or along cemented joints difficult. The only shear orientations which are not artificially strengthened by intersecting the platens have to be inclined at more than $45^o$ to the $\sigma_1$ direction, and by definition to be incapable of shear failure under the Mohr-Coulomb criterion. Gehring therefore manages to reduce the effects of all 3 types of variability (Mode of failure, Size and Shape), and is able to use analogies with past experience in interpreting measured uniaxial compressive strengths on samples from a new job site. His approach is strictly "non-standard", and is not one that a public testing laboratory can often follow.

Dr. Gehring also measures the failure energy of his test specimens, expressed as the area under a plot of axial force versus axial deformation, up to the point of strength failure. See Figure 1 : Gehring (pers. comm.) He attaches great significance to measuring both the elastic and the non-elastic deformation. By integrating the area under the stress/strain curve he obtains a Fracture Energy value in Newton.metres. He also obtains Specific Fracture Energy, Nm/MPa, by dividing the Fracture Energy by the Uniaxial Compressive Strength. These 2 strength parameters provide a useful basis for him to be able to estimate the field

**Figure 1Load-Deformation Diagram**
**(K. Gehring)**

performance of his employer's tunnel excavating and mining machines. The values are only strictly comparable with other values determined in the same laboratory with the same apparatus. In this sense they are similar to other private or proprietary tests carried out by other machine manufacturers : non-standard tests, carried out at only one location in the world, only validly interpreted by one person or organisation. They are not therefore in the spirit of test procedures standardised by the ASTM and the ISRM, which are intended to be transparent and universally replicable.

### 3.1 Rock Toughness Index

This author, after discussions with Dr. Gehring, decided to modify his reporting procedures, to make them applicable to the wider range of specimen sizes which come into my laboratory and also dimensionally exact.

Firstly, the values of Fracture Energy and Specific Fracture Energy are reported as-measured, and also adjusted to the values which should have been measured in a 50∗50mm cylinder of the same rock, by multiplying the measured Fracture Energy by the ratio of the volume of 50∗50mm cylinder to the volume of the actual test specimen. This is to enable the production of "standardised" values, easily comparable with the large body of past experience.

Secondly, it is more usual to produce stress/strain curves than force/deformation curves (and the former are independent of specimen dimensions). Whereas Gehring integrates the area under the test curve to get Fracture Energy in Nm, the comparable integration of the area under the stress/strain curve gives an area in MPa, or $N/m^2$, or $ML^{-1}T^{-2}$. This is dimensionally equivalent to Joules per cubic metre i.e. energy Nm divided by volume $m^3 = N/m^2$ = stress, or unit energy $ML^2T^{-2}$ divided by unit volume $L^3 = ML^{-1}T^{-2}$. This laboratory value of strain energy per unit volume, or Specific Energy, should be directly correlatable with field specific energy of cutting or breaking, for particular excavation systems. By analogy with Gehring's Specific Fracture Energy, Nm/MPa (which has the intractable dimensions of $10^{-6}*L^3$ or cubic centimetres), this author decided to divide the laboratory Strain Energy at Failure per unit volume (or Laboratory Specific Energy) by the Uniaxial Compressive Strength, to get a dimensionless number which is called Rock Toughness Index. The convenient unit for reporting Laboratory Specific Energy is $kJ/m^3$, and the convenient unit for reporting Uniaxial Compressive Strength is $MN/m^3$ or MPa, so that Rock Toughness Index is defined as 1000∗Laboratory Specific Energy($kJ/m^3$)/Uniaxial Compressive Strength (MPa).

Low values of Rock Toughness Index will be produced by stiff brittle rocks, which exhibit little if any non-elastic deformation before failure. A linear-elastic rock having a Modulus Ratio (Young's Modulus E/Uniaxial Compressive Strength) value of greater than 500 is defined as "High Modulus Ratio", and will have a Rock Toughness Index of less than 1.0 A linear-elastic rock having a Modulus Ratio value of less than 200 is defined as "Low Modulus Ratio", and will have a Rock Toughness Index of greater than 2.5 A linear-elastic rock having a Modulus Ratio value of between 200 and 500 is defined as "Normal Modulus Ratio", and will have a Rock Toughness Index of between 1.0 and 2.5 In each case the Rock Toughness Index will be increased by the presence of non-elastic or plastic deformation before failure.

It is therefore suggested that estimations of excavatability of a rock, from its uniaxial compressive strength, may be rendered more valid by consideration also of the Rock Toughness Index. If a rock has a Rock Toughness Index of between 1.0 and 2.5 it may be regarded as "normal", and past experience with rocks having comparable strengths may be used as a guide to predictions of excavatability. If the rock has a Rock Toughness Index greater than 2.5 this will indicate that the rock will absorb an abnormal amount of strain energy before it will fail, so it could be tougher to excavate than might be predicted from consideration only of its uniaxial compressive strength.

Figure 2 is a typical example of a recent test curve.

### 3.2 Excavatability From Fracture Energy

Farmer (1986) showed correlations between volume excavation rates for tunnelling machines and a property called "Fracture Toughness", defined by him as equivalent to $(\sigma_c)^2/E$.

See Figures 3 and 4, reproduced from Farmer (1986).

In fact this quantity is equal to twice the strain energy per unit volume at failure in uniaxial compression, or Specific Energy in the sense used above : stored elastic strain energy = $\sigma^2/2E$ per unit volume.

The term "Fracture Toughness", as generally used, is a measure of the inherent tensile strength of a substance,

**Figure 2U.C.S. Test Stress/Strain Plot**

and can not be calculated from the uniaxial compressive strength.

Farmer's approach seemed to this author to be worth combining with Gehring's approach : the former developing predictor equations from Specific Energy (albeit with the assumption that the rock behaviour would be linear elastic up till failure), the latter measuring both elastic and non-elastic or plastic deformation of the rock up till failure. Farmer's published data points were converted to equivalent values of elastic strain energy per unit volume (Specific Energy), and curve fitting was done on these points so that field excavation rates can be predicted from the Strain Energy at Failure measured in the uniaxial

**Figure 3(From Farmer, 1986)**

**Figure 4(From Farmer, 1986)**

compression test. There may be some doubt as to the validity of my use of Farmer's published data in this way, as his assumed E values may have been linear "best-fits" to a nonlinear curve, and the area beneath the actual curve as measured by Gehring and me may have been greater than that below Farmer's assumed straight line. However, this approach will become more refined, as more data on field excavation rates are reported, and compared with the Specific Energy measured in the uniaxial compression tests.

The Rock Toughness Index, representing as it does the ratio between the Specific Energy and the Uniaxial Compressive Strength, may prove a simple mental short cut for the excavation engineer to quickly assess the feasibility of using any particular machine for rock cutting. The establishment of correlations between mining and tunnelling machine excavation rates and the corresponding Rock Toughness Index should now have high priority.

## 4.1 Tunnel Boring

The performance of tunnel boring machines may be predicted by a method developed by the University of Trondheim, Norway. (Johannessen, 1988). The input data for this model include some specialised rock tests, such as the Swedish Brittleness Index, the Sievers J-number, Norwegian Abrasion Value, etc. as well as the rock petrography, and structural geological information.

Rock Toughness Index values, compared with some recent predictions made by the Norwegian method, are shown on Figure 5. It must be emphasised that the TBM advance rates shown are predictions only, and data from the tunnel as actually constructed will be needed to verify the correlations. However, the positive

**Figure 5Predicted TBM Performance**

correlation indicates that Rock Toughness Index could eventually be used as a predictor of Tunnel Boring Machine performance.

## 4.2 Roadheader Cutting Rates

The performance of roadheaders may be predicted by several methods. McFeat-Smith & Fowell (1979) showed how field cutting rates of medium and heavy weight roadheaders in massive rock could predicted from 2 rock properties measured in the laboratory : the N.C.B. Cone Indenter Hardness and the Plasticity Coefficient. The correlation between field cutting rates predicted by me using this method, for a Typical heavy weight roadheader, and the Rock Toughness Index values for the same rocks, show a general trend for a decrease in field cutting rate with increase in Rock Toughness Index, similar to that shown in Figure 6.

The method used by Voest-Alpine (Gehring, pers. comm.) uses measured Uniaxial Compressive Strength and Brazilian indirect tensile strength as the main laboratory strength values as input data for the equations calculating performance of their roadheaders. Figure 6 shows the correlation between field production rates predicted by this method, for an AM105 roadheader, and the Rock Toughness Index values for the same rocks. There is a general trend for a decrease in production rate with increase in Rock Toughness Index.

The correlation coefficients are low, because of the scatter of data points, and the examples are for predicted performances only. Data from actual roadheader production are required, to verify the hypothesis that Rock Toughness Index may prove to be a useful quantative or semi-quantitative predictor of roadheader performance, or another factor in the predictor equations.

**Figure 6 Predicted Roadheader Performance**

### 4.3 Rock Bursts

The Rock Toughness Index may also be useful in prediction of rock bursts, or at least the identification of rocks which may have a greater than average propensity to exhibit rockbursting when highly stressed.

Rockbursts have given trouble in deep level mining in hard strong rocks in countries such as India, Canada and South Africa, causing loss of life and damage to and collapse of tunnels and stopes. The rockburst problem is now being taken seriously in Australia, with deep mines in high stress environments suffering explosive rock failures. There is a need for a technique for identifying, in the feasibility study or planning stage, any rocks which may be prone to rockbursting, so that the extra costs of measures to cope with rockbursts can be factored in.

The Rock Toughness Index measures the amount of strain energy in a rock at the moment that it breaks.

It may consequently be then applicable to the 2 quite separate problems :

(1) How much strain energy has to be artificially put into the rock by the excavation machine to break it?;

(2) How much strain energy will be released by the rock after it breaks naturally, to be dealt with by the mining or tunnelling engineer?

### 4.4 Fracture Toughness

Determination of rock properties thought likely by mining geotechnical consultants to be significant for planning of new mines in rockburst-prone rock masses has included both Fracture Toughness and Rock Toughness Index determinations. Fracture Toughness, the method suggested by some consultants as having most promise, was determined by the Chevron-Notch Short Rod method. Figure 7 shows a scatter diagram of

**Figure 7Fracture Toughness &**
**Rock Toughness Index**

the correlations between the 2 properties. The correlation coefficient is low, but shows a trend of increasing Fracture Toughness with increasing Rock Toughness Index. It is felt that more work is justified to investigate this correlation, and also to carry out extensive tests with both methods on rock samples from any field site which suffers rock bursts in the future. The simplicity of specimen preparation for Rock Toughness Index determination, compared with that for Fracture Toughness determination, means that the former test could be carried out in greater numbers, and used to indicate special zones from which to select specimens for Fracture Toughness testing.

### 5.1 Conclusion

Rock Toughness Index is a function of the Uniaxial Compressive Strength and the Specific Energy of fracture of a rock. It is relatively easy to determine in a testing laboratory, and could become a useful indicator of the relative toughness of rocks, in 2 quite different practical situations :

(1) In excavating them by mining or tunnelling machinery;

(2) In their undergoing violent failure from the roof or wall of an underground excavation.

Comparisons of excavation performance with the Rock Toughness Index are needed to validate its usefulness.

## 5.2 References


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