UNDERGROUND MINE STABILITY

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Subjects

1. Rock mass structure and characterisation
2. In situ and induced stress
3. Rock mass properties
4. Rock mass classification
5. Underground excavation failure mechanism
6. Instrumentation

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Rock Mass Structure and Characterisation

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Introduction

Rock material is the term used to describe the intact rock between discontinuities; it might be represented by a hand specimen or piece of drill core examined in the laboratory.

Rock mass is the total in situ medium containing structural features.

Major Types Of Structural Features

- Bedding planes
- Folds
- Faults
- Shear zones
- Dykes
- Joints
- Vein

Suggested methods for the quantitative description (Barton-ISRM, 1978)

1. Orientation
2. Spacing
3. Persistence
4. Roughness
5. Wall strength
6. Aperture
7. Filling
8. Seepage
9. Number of sets
10. Block size
11. Drill core

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Rock Quality Designation (RQD)

In classifying rock masses for engineering purposes, it is common practice to quote values of **Rock Quality Designation (RQD)**, a concept introduced by Deere (1964, 1968) in an attempt to quantify discontinuity spacing.

Rock Quality Designation (RQD)

RQD is determined from drill core and is given by

$$RQD = \frac{100 \sum x_i}{L}$$

where x_i are the lengths of individual pieces of core in a drill run having lengths of 0.1 m or greater and L is the total length of the drill run.

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Rock Quality Designation (RQD)

Priest and Hudson (1976) found that an estimate of RQD could be obtained from discontinuity spacing measurements made on core or an exposure using the equation

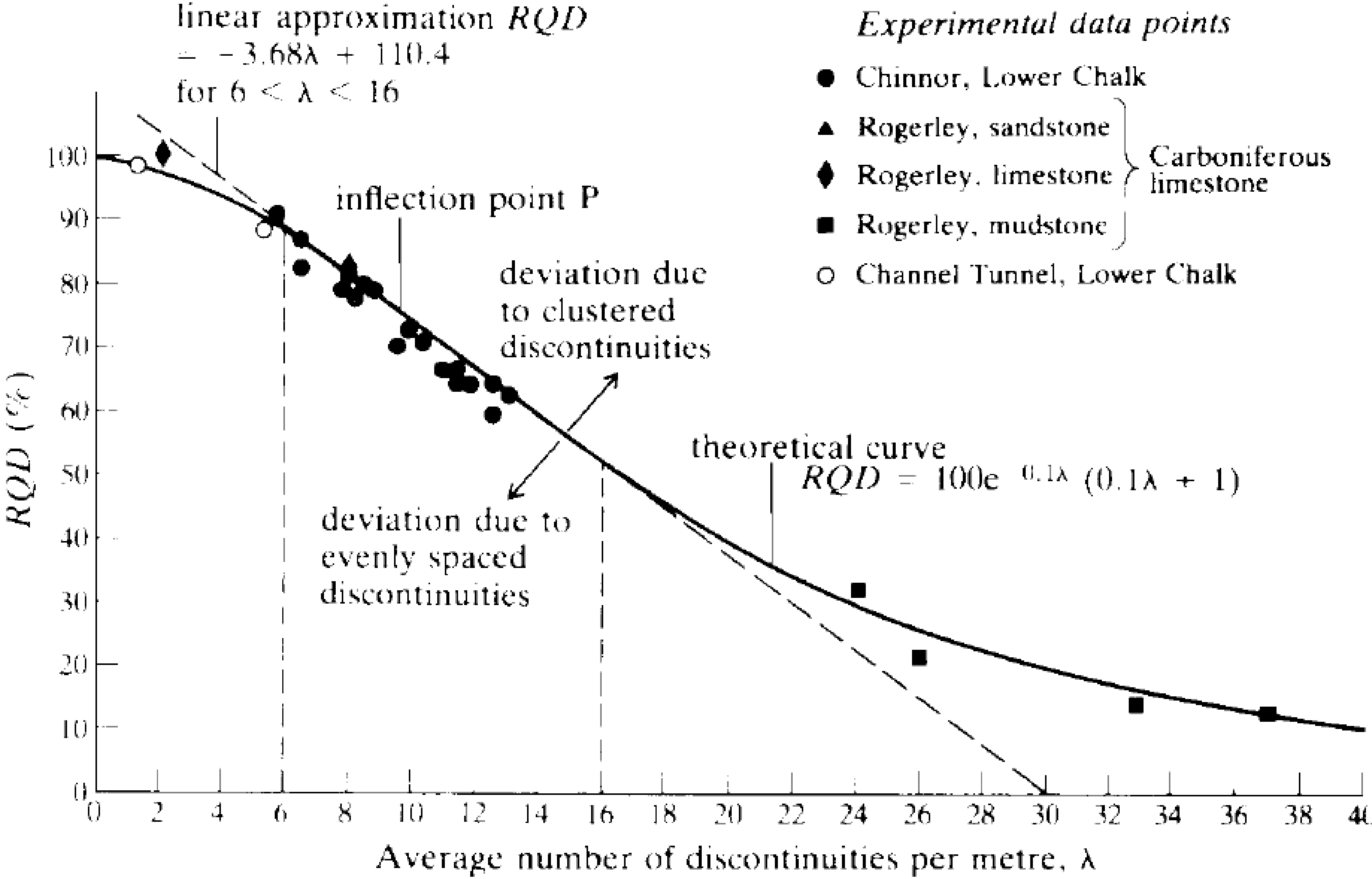
$$RQD = 100e^{-0.1\lambda}(0.1\lambda + 1)$$

where λ is average number of discontinuities **per meter**

Rock Quality Designation (RQD)

For values of λ in the range 6 to 16/m, a good approximation to measured RQD values was found to be given by the linear relation

$$RQD = -3.68\lambda + 110.4$$



Relation between RQD and mean discontinuity frequency (after Priest and Hudson, 1976)

Introduction

Mapping of geological structure is an essential component of the design of underground excavations.

Data collected from the mapping of these structures are used to determine the orientation of the major joint sets and to assess the potential modes of structural failure.

Engineering Geological Data Collection

Standardised approaches to the collection of engineering geology data, for civil and mining engineering purposes, have been proposed by the Geological Society of London (1977) and by the International Society of Rock Mechanics (ISRM, 1978).

Engineering Geological Data Collection

The main goal in **engineering geological data collection** is to be able to describe the rock mass as accurately as possible. This will assist in the determination of a **rock mass classification** as well as providing a means of communication between geologists and engineers working together on a project.

IN SITU STRESS

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Rock at depth is subjected to **stresses** resulting from **the weight of the overlying strata** and from **locked in stresses of tectonic origin**.

VERTICAL STRESS

Consider an element of rock at a depth z below the surface. **The weight of the vertical** column of rock resting on this element is **the product of the depth and the unit weight of the overlying rock mass**. This stress is estimated from the simple relationship:

$$\sigma_v = \gamma z$$

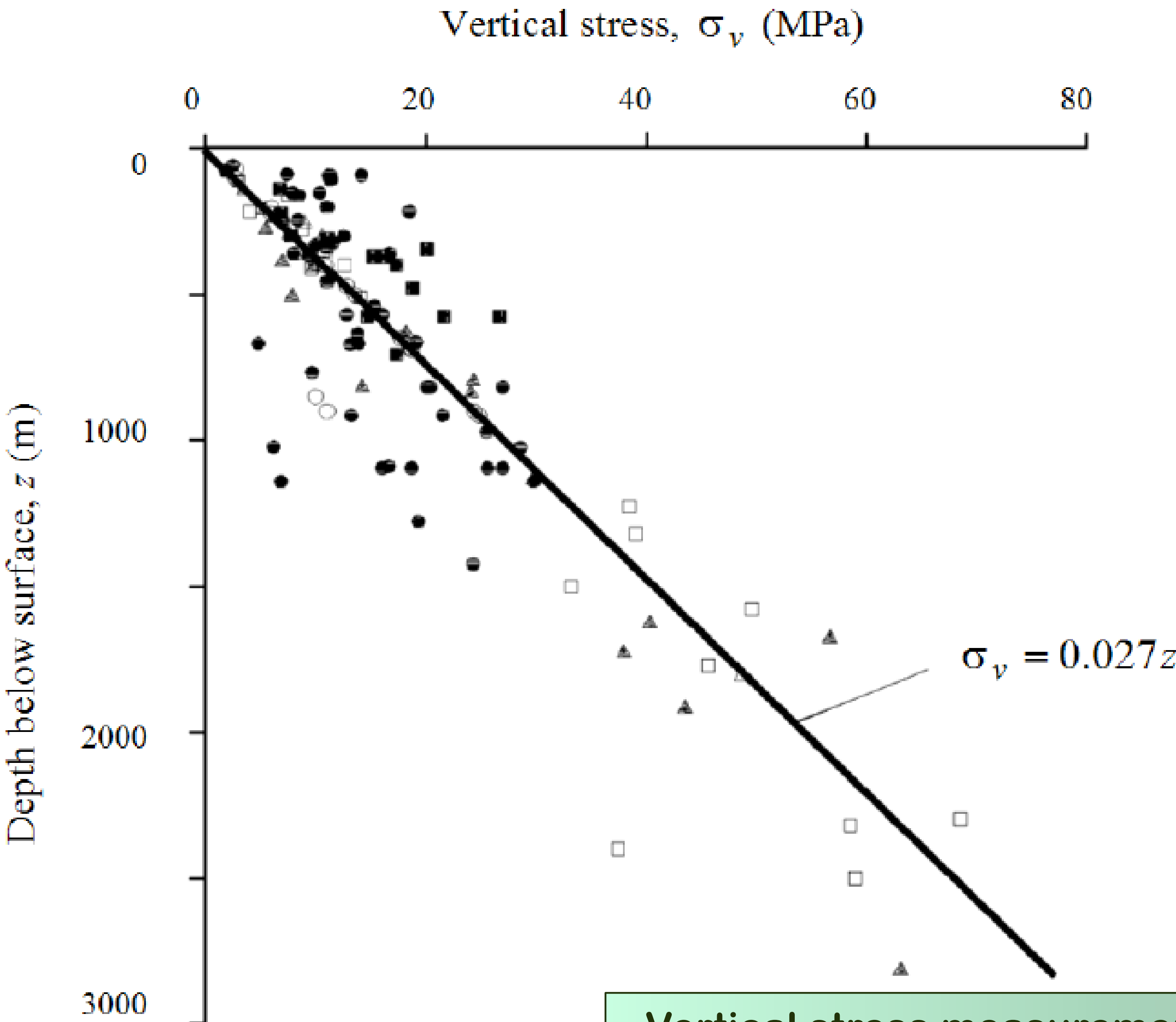
where σ_v is the vertical stress;

γ is the unit weight of the overlying rock; and

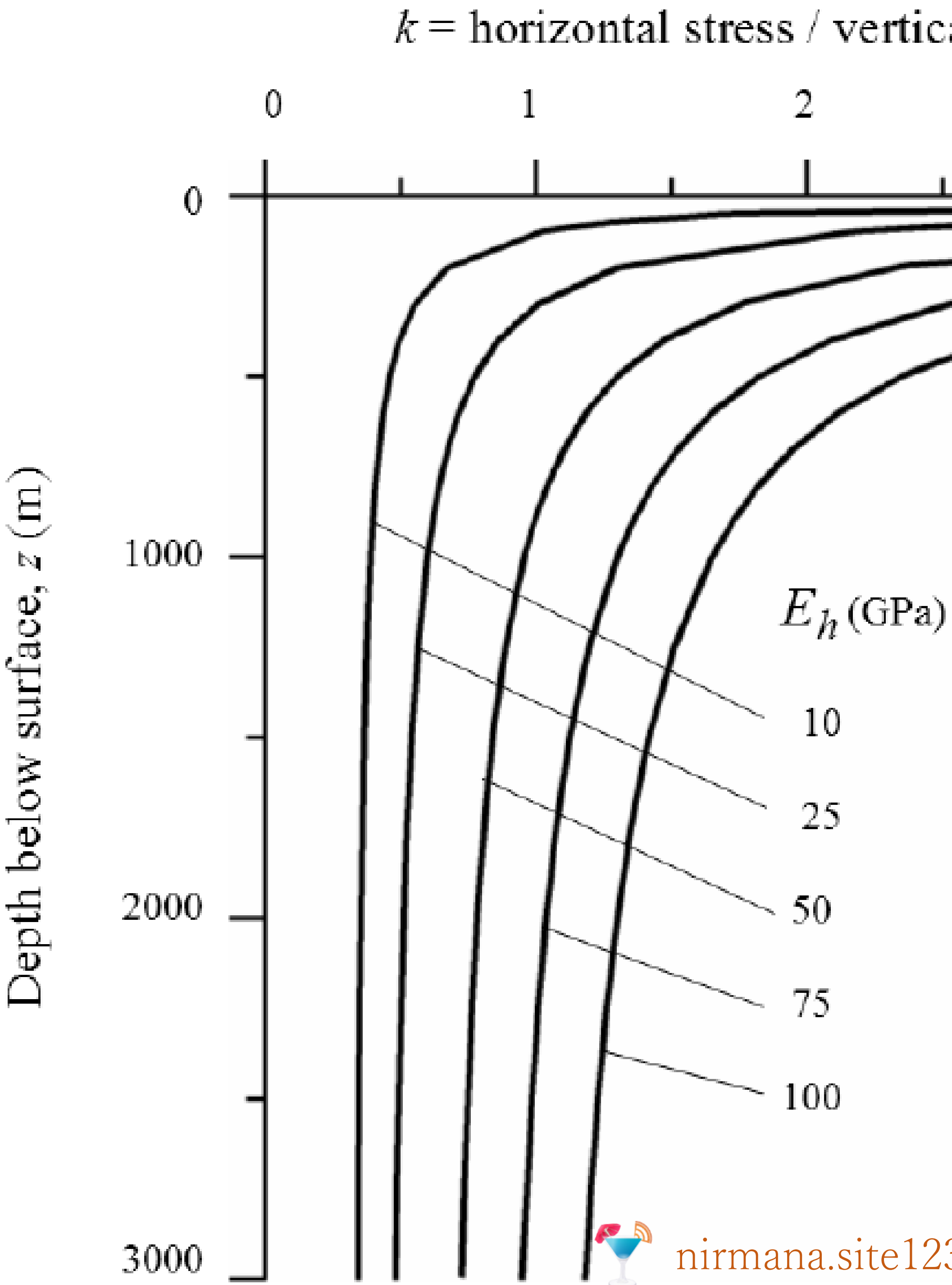
z is the depth below surface.

VERTICAL STRESS

Measurements of vertical stress at a various mining and civil engineering sites around the world confirm that this relationship is valid although, as illustrated in Figure 1, there is a significant amount of scatter in the measurements.



Vertical stress measurements from mining and civil engineering projects around the world (After Brown and Hoek, 1978)



Ratio of Horizontal to Vertical Stress for Different Deformation Moduli Based Upon Sheorey's Equation (After Sheorey, 1994)

HORIZONTAL STRESS

The horizontal stresses acting on an element of rock at a depth z below the surface are much more complex to estimate than the vertical stresses. Normally, **the ratio of the average horizontal stress to the vertical stress is denoted by the letter** k such that:

$$\sigma_h = k\sigma_v = k\gamma z$$

$$k = \frac{\sigma_h}{\sigma_v}$$

1952

Terzaghi and Richart suggested that, for a gravitationally loaded rock mass in which no lateral strain was permitted during formation of the overlying strata, the value of k is independent of depth and is given by $k = v/(1 - v)$, where v is the Poisson's ratio of the rock mass.

This relationship <u>was widely used</u> in the early days of rock mechanics but, as discussed next, it proved to be <u>inaccurate</u> and <u>is seldom used today</u>.

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$$k = \frac{\sigma_h}{\sigma_v}$$

1994

Sheorey did provide a simplified equation which can be used for estimating the horizontal to vertical stress ratio k. This equation is:

$$k = 0.25 + 7E_h\left(0.001 + \frac{1}{z}\right)$$

where z (m) is the depth below surface and E_h (GPa) is the average deformation modulus of the upper part of the earth's crust measured in a horizontal direction.

- **Special caution in layered sedimentary rocks**

  (Why?)

IN SITU STRESS

As pointed out by Sheorey, his work does not explain the occurrence of measured vertical stresses that are higher than the calculated overburden pressure, the presence of very high horizontal stresses at some locations, or why the two horizontal stresses are seldom equal.

These differences are probably due to **local topographic** and **geological features** that cannot be taken into account in a large scale model such as that proposed by Sheorey (Hoek, 2006).

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IN SITU STRESS

Where sensitivity studies have shown that the in situ stresses are likely to have a **significant influence** on the behaviour of underground openings, it is recommended that **the in situ stresses should be measured**.

Case Study

1. Consider an element of rock at a depth of 1,000 m below the surface. The unit weight of the overlying rock mass typically about 2.7 tonnes/m^3. **Estimate the vertical in situ stress.**
2. From laboratory test, it is indicated that the Young's modulus has an average of 5,600 MPa. Based on a large scale model that proposed by Sheorey (1994), **estimate the horizontal in situ stress.**

Solution

1. $\sigma_v = \gamma z = \rho g z = 27\ MPa$

2. $k = 0.25 + 7E_h\left(0.001 + \frac{1}{z}\right) = 0{,}3284$

   $\sigma_h = k\sigma_v = 8{,}87\ MPa$

Methods of In Situ Stress Determination

1. Triaxial strain cell
2. Flatjack measurements
3. Hydraulic fracturing
4. Kaiser effect

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World Stress Map

The natural state of stress near the earth's surface is of world-wide interest, from the points of view of both industrial application and fundamental understanding of the geomechanics of the litosphere. From observations of the natural state of stress in many separate domains of the litosphere, **world stress map** has been assembled to show the relation between the principal stress directions and the megascopic structure of the earth's crust.

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World Stress Map

The value of such a map in mining rock mechanics is that it presents some high level information on orientations of the horizontal components of the pre-mining principal stresses which *can be incorporated* in site investigations and preliminary design and scoping studies.

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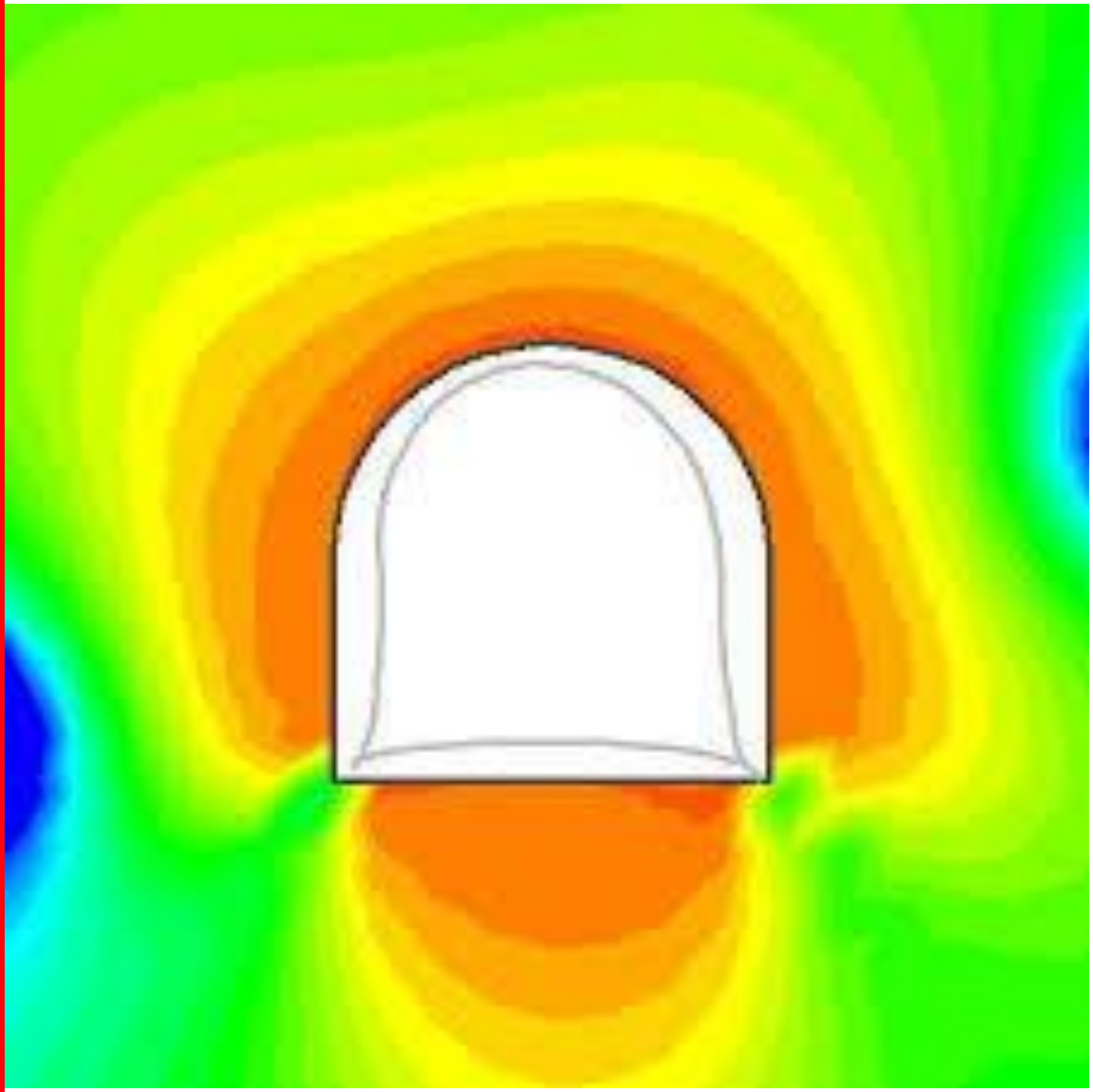

Induced Stress

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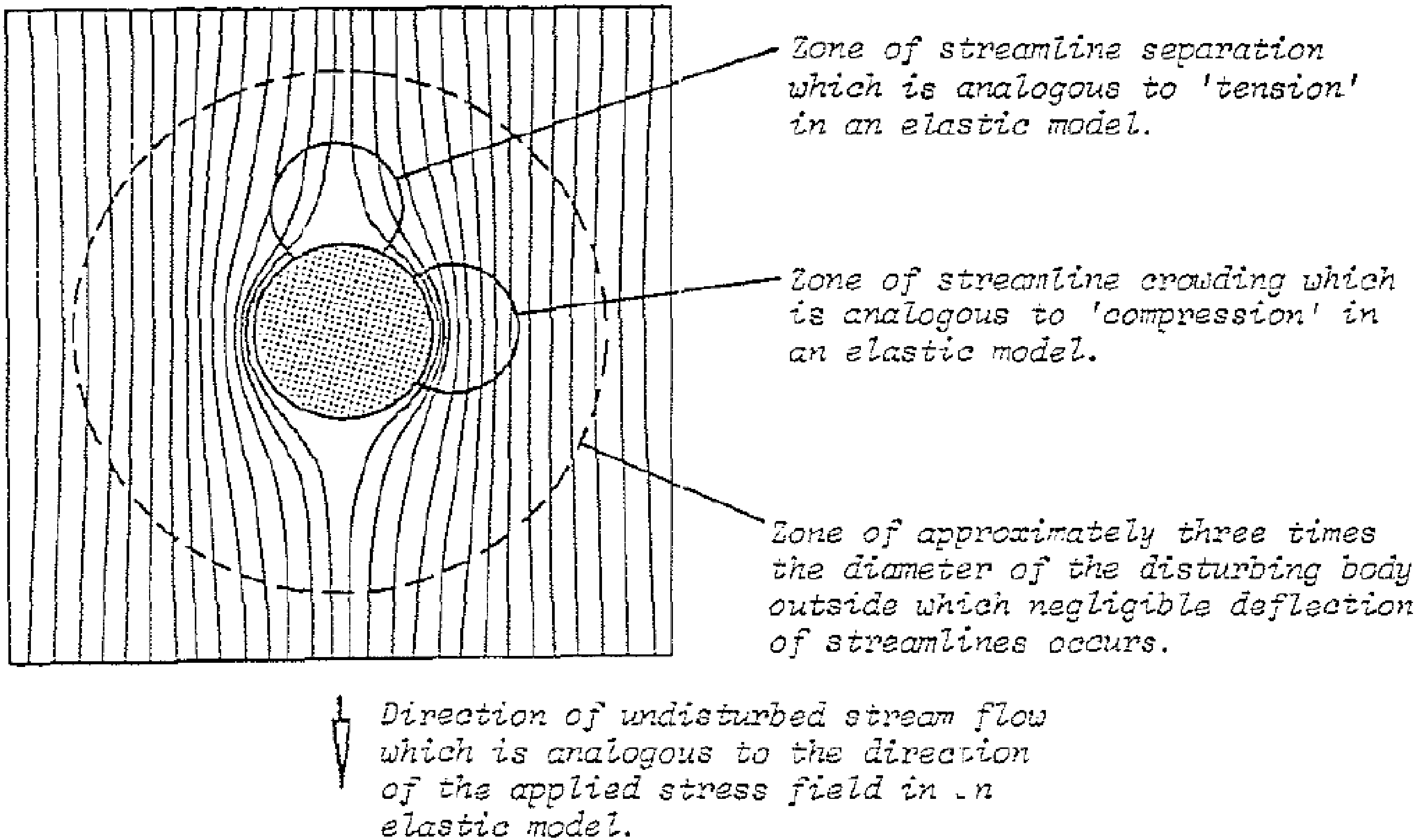
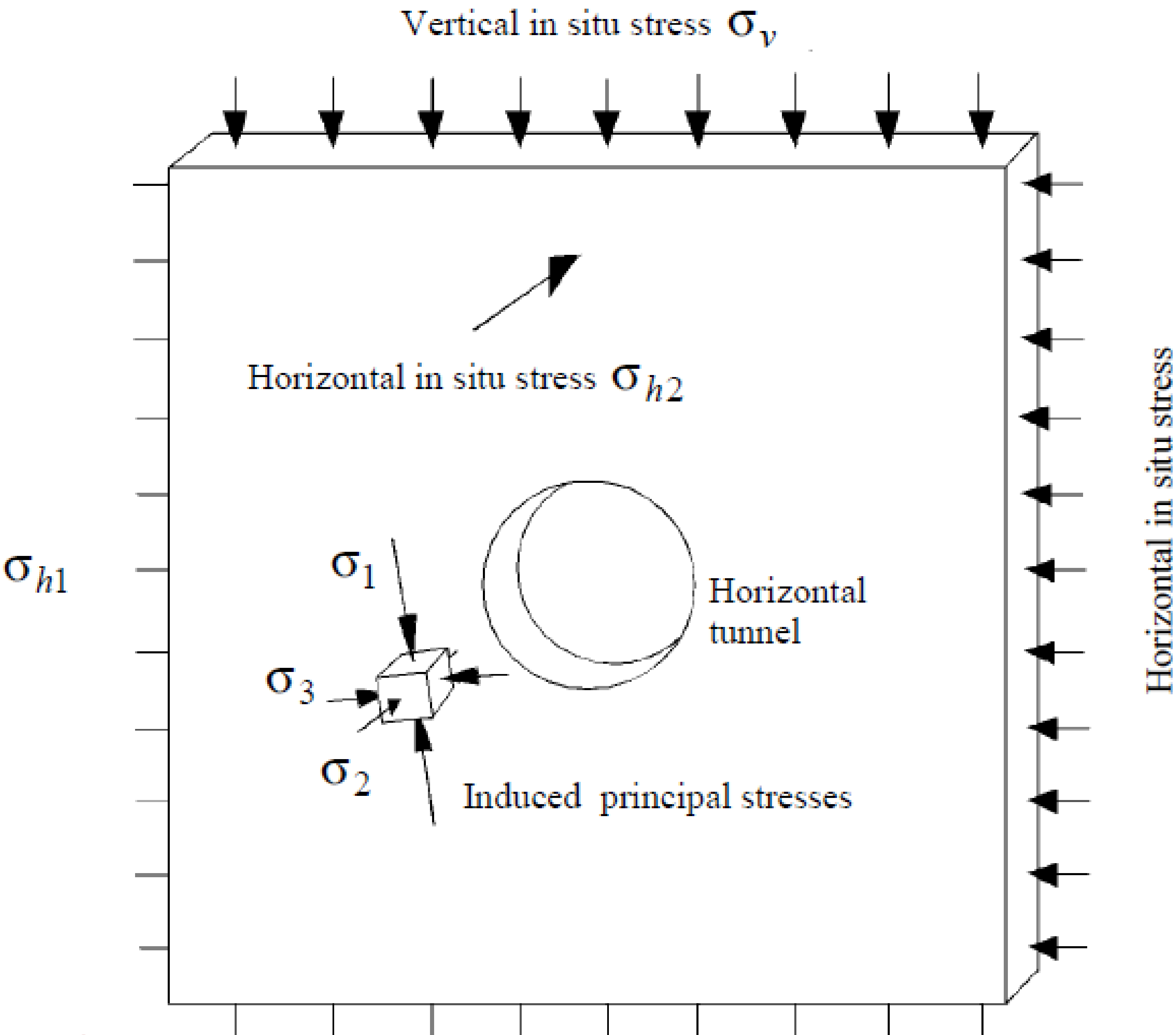
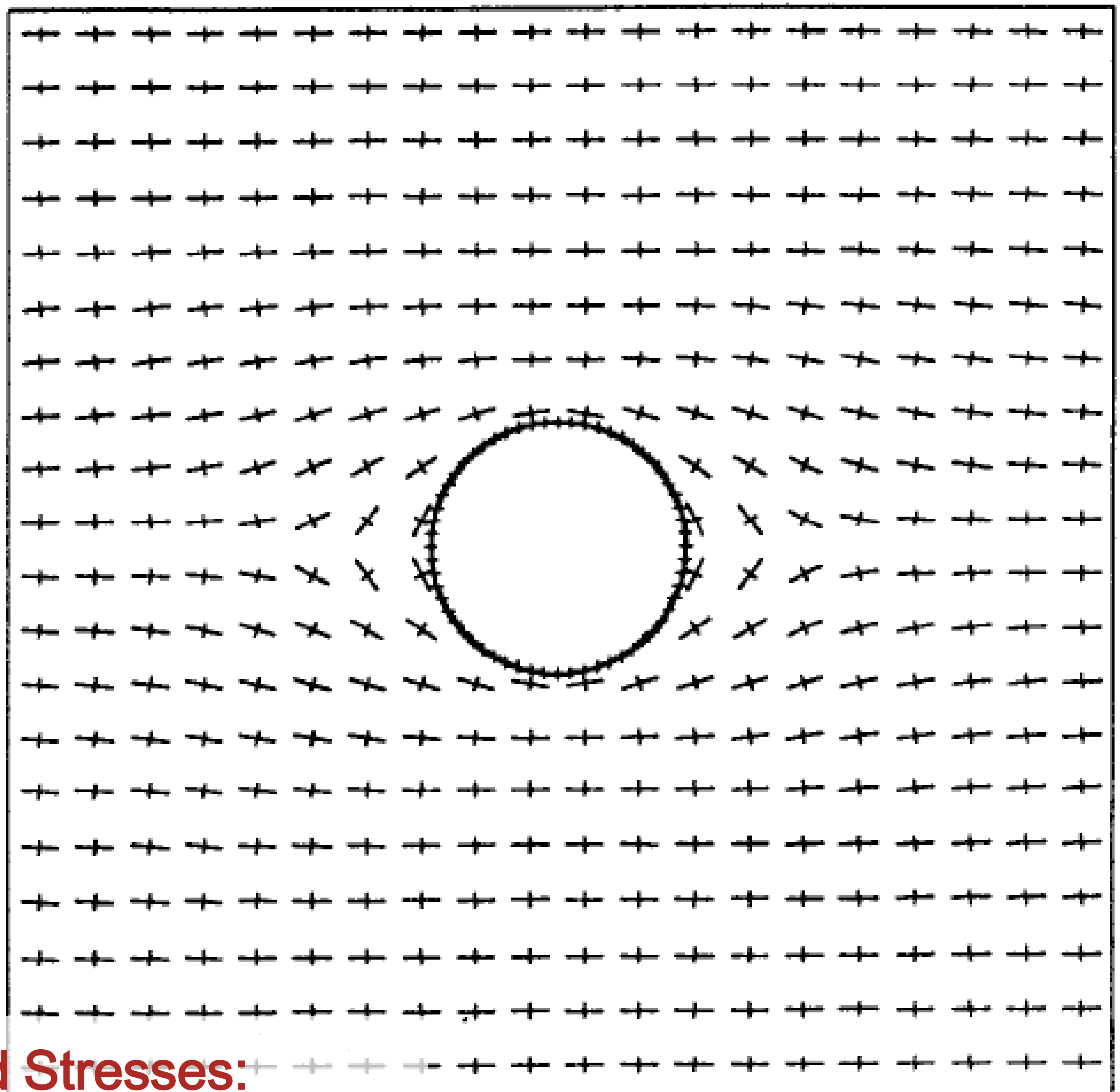


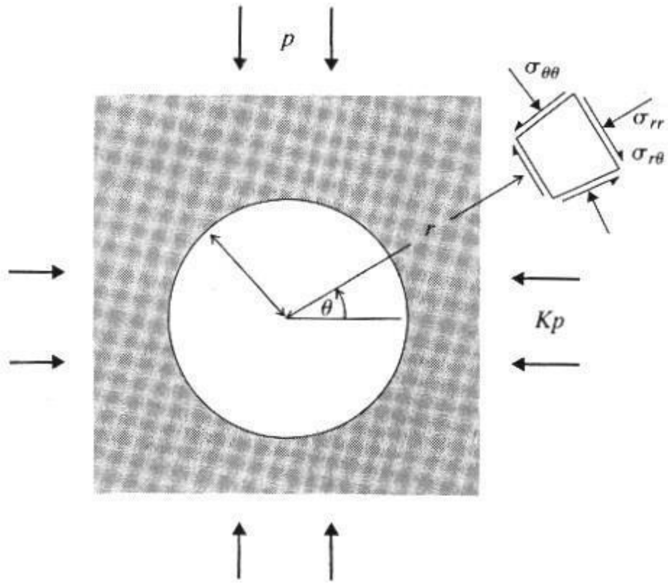
Deflection of Streamlines Around a Cylindrical Obstruction
(Hoek, E. & Brown, E.T. *Underground Excavations in Rock*. 1980)



Induced Stresses: Closed Form Solutions



Induced Stresses: Closed Form Solutions



CIRCULAR TUNNEL

$$\sigma_{rr} = \frac{p}{2}\left[(1+K)\left(1-\frac{R^2}{r^2}\right)-(1-K)\left(1-4\frac{R^2}{r^2}+3\frac{R^4}{r^4}\right)\cos 2\theta\right]$$

$$\sigma_{\theta\theta} = \frac{p}{2}\left[(1+K)\left(1+\frac{R^2}{r^2}\right)+(1-K)\left(1+3\frac{R^4}{r^4}\right)\cos 2\theta\right]$$

$$\sigma_{r\theta} = \frac{p}{2}\left[(1-K)\left(1+2\frac{R^2}{r^2}-3\frac{R^4}{r^4}\right)\sin 2\theta\right]$$

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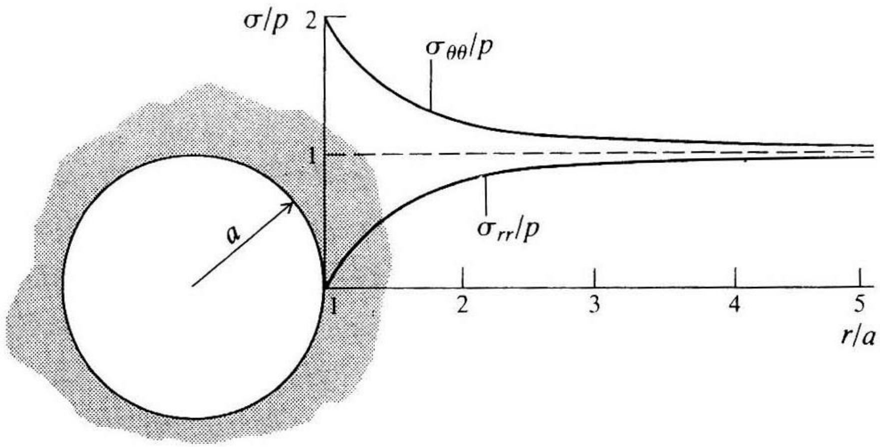
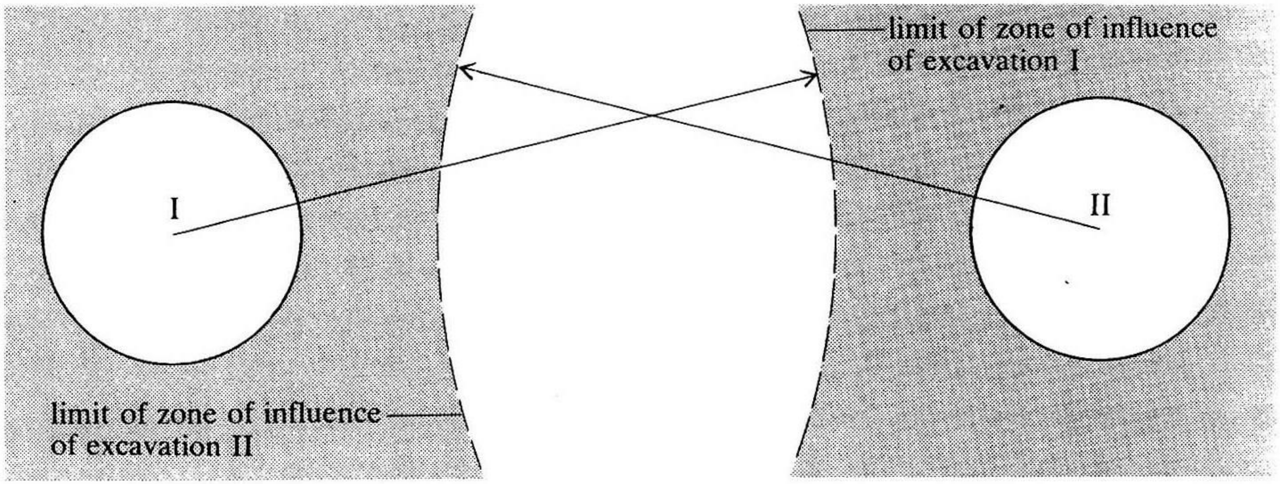
Circular Tunnel: Stress Zone of Influence





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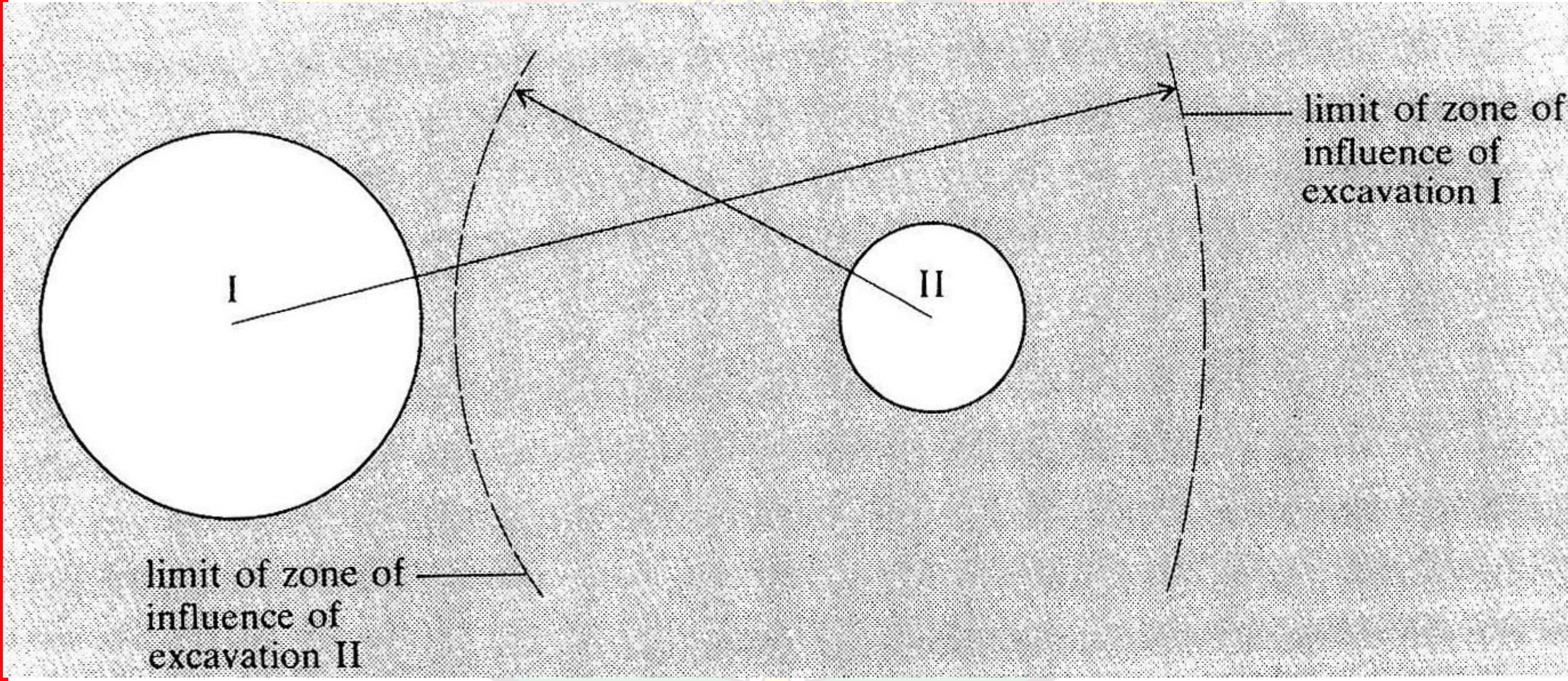
Circular Tunnel: Stress Zone of Influence



A Practical Method to Estimate The Magnitude of The Tangential Stresses (Hoek & Brown, 1980)

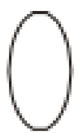
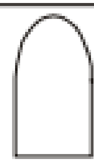
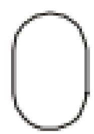
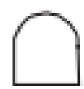
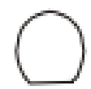
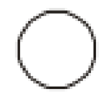
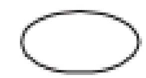
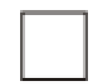
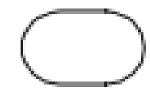
- The tangential stress in roof $\sigma_{\theta r} = (A \times k - 1)p_z$
- The tangential stress in wall $\sigma_{\theta w} = (B - k)p_z$

A and B are roof and wall factors for various excavation shapes;

k is the ratio horizontal/vertical stress;

p_z is the vertical in situ stress.

A Practical Method to Estimate The Magnitude of The Tangential Stresses (Hoek & Brown, 1980)

VALUES OF CONSTANTS A & B									
									
A	5.0	4.0	3.9	3.2	3.1	3.0	2.0	1.9	1.8
B	2.0	1.5	1.8	2.3	2.7	3.0	5.0	1.9	3.9

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ROCK MASS PROPERTIES

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Reliable estimates of the strength and deformation characteristics of rock masses are required for almost any form of analysis used for the design of slopes, foundations, and underground excavations.

Hoek and Brown (1980) proposed a method for obtaining estimates of **the strength of jointed rock masses**, based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks.

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The application of the method to very poor quality rock masses required further changes (Hoek, Wood, and Shah-1992) and, eventually, the development of a new classification called the **Geological Strength Index, GSI** (Hoek, Kaiser, and Bawden-1995, Hoek-1994, Hoek and Brown-1997, Hoek, Marinos and Benissi-1998, Marinos and Hoek-2001).

Generalized Hoek-Brown Criterion

The Generalized Hoek-Brown failure criterion for **jointed rock masses** is defined by:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b\frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

Generalized Hoek-Brown Criterion

$$\sigma_1{}' = \sigma_3{}' + \sigma_{ci}\left(m_b \frac{\sigma_3{}'}{\sigma_{ci}} + s\right)^a$$

$\sigma_1{}'$ and $\sigma_3{}'$ are the maximum and minimum effective principal stresses at failure,

m_b is the value of the Hoek-Brown constant m for the rock mass

s and a are constants which depend upon the rock mass characteristics, and

σ_{ci} is the uniaxial compressive strength of the intact rock pieces.

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Generalized Hoek-Brown Criterion

In order to use the Hoek-Brown criterion for estimating the strength and deformability of jointed rock masses, three *properties* of the rock mass have to be estimated. These are:

- ✓ uniaxial compressive strength (σ_{ci}) of the intact rock pieces,
- ✓ value of the Hoek-Brown constant (m_i) for these intact rock pieces, and
- ✓ value of the Geological Strength Index (GSI) for the rock mass.

Generalized Hoek-Brown Criterion

$$m_b = m_i exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

Intact Rock Properties

$$\sigma_1{}' = \sigma_3{}' + \sigma_{ci}\left(m_i\frac{\sigma_3{}'}{\sigma_{ci}} + 1\right)^{0.5}$$

The relationship between the principal stresses at failure for a given rock is defined by two constants, the uniaxial compressive strength (σ_{ci}) and a constant (m_i).

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Intact Rock Properties

Wherever possible the values of these constants should be determined by *statistical analysis* of the results of a set of triaxial tests on carefully prepared core samples.

Intact Rock Properties

Note that the range of minor principal stress ($\sigma_3{}'$) values over which these tests are carried is critical in determining reliable values for the two constants. In deriving the original values of σ_{ci} and m_i, Hoek and Brown (1980) used a range of $\mathbf{0} < \boldsymbol{\sigma_3'} < \mathbf{0.5}\boldsymbol{\sigma_{ci}}$ and, in order to be consistent, it is essential that the same range be used in any laboratory triaxial tests on intact rock specimens. *At least five* well spaced data points should be included in the analysis.

Intact Rock Properties

Once the five or more triaxial test results have been obtained, they can be analysed to determine the uniaxial compressive strength σ_{ci} and the Hoek-Brown constant m_i as described by Hoek and Brown (1980).

Intact Rock Properties

In this analysis, equation $\sigma_1{}' = \sigma_3{}' + \sigma_{ci}\left(m_i \frac{\sigma_3{}'}{\sigma_{ci}} + 1\right)^{0.5}$

is re-written in the form:

$$y = m\sigma_{ci}x + s\sigma_{ci}$$

where $x = \sigma_3{}'$ and $y = (\sigma_1{}' - \sigma_3{}')^2$

Intact Rock Properties

For n specimens the uniaxial compressive strength (σ_{ci}), the constant m_i, and the coefficient of determination r^2 are calculated from:

$$\sigma_{ci}{}^2 = \frac{\sum y}{n} - \left[\frac{\sum xy - (\sum x \sum y / n)}{\sum x^2 - ((\sum x)^2 / n)}\right] \frac{\sum x}{n}$$

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Intact Rock Properties

$$m_i = \frac{1}{\sigma_{ci}}\left[\frac{\sum xy - (\sum x \sum y/n)}{\sum x^2 - ((\sum x)^2/n)}\right]$$

$$r^2 = \frac{[\sum xy - (\sum x \sum y/n)]^2}{[\sum x^2 - ((\sum x)^2/n)][\sum y^2 - ((\sum y)^2/n)]}$$

Note that **high quality triaxial test data** will usually give a coefficient of determination r^2 **of greater than 0.9**.

EXERCISE

Analysis of triaxial test data

σ_3 (MPa)	σ_1 (MPa)
0	38.3
5	72.4
7.5	80.5
15	115.6
20	134.3

Intact Rock Properties

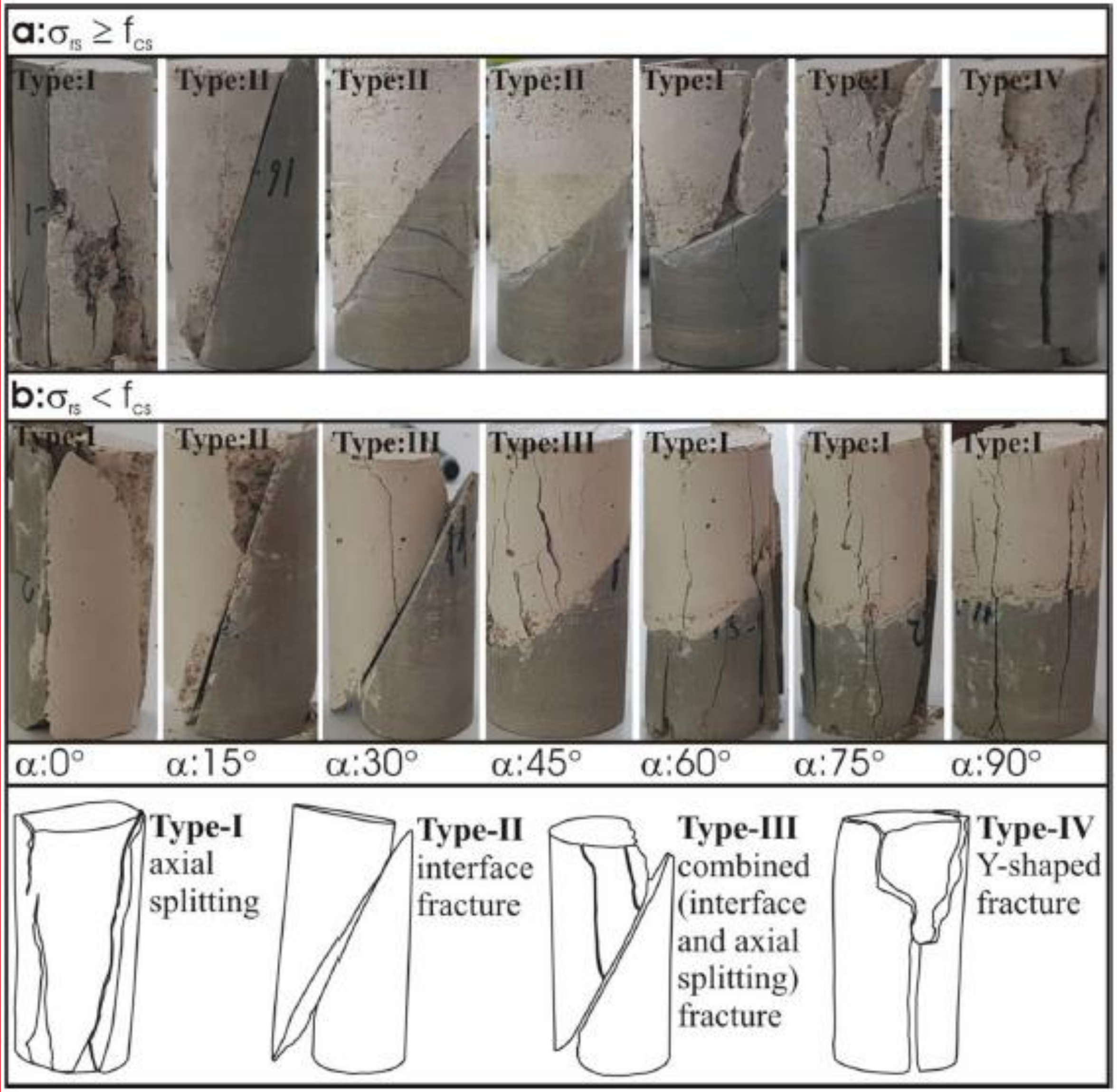
These calculations, together with many more related to the Hoek-Brown criterion can also be performed by the program RocLab that can be downloaded (free) from www.rocscience.com.

Values of the constant m_i for intact rock, by rock group (Hoek, 2007)

Rock type	Class	Group	Texture: Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates* (21 ± 3) Breccias (19 ± 5)	Sandstones 17 ± 4	Siltstones 7 ± 2 Greywackes (18 ± 3)	Claystones 4 ± 2 Shales (6 ± 2) Marls (7 ± 2)
SEDIMENTARY	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestones (10 ± 2)	Micritic Limestones (9 ± 2)	Dolomites (9 ± 3)
SEDIMENTARY	Non-Clastic	Evaporites		Gypsum 8 ± 2	Anhydrite 12 ± 2	
SEDIMENTARY	Non-Clastic	Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4) Metasandstone (19 ± 3)	Quartzites 20 ± 3	
METAMORPHIC	Slightly foliated		Migmatite (29 ± 3)	Amphibolites 26 ± 6		
METAMORPHIC	Foliated**		Gneiss 28 ± 5	Schists 12 ± 3	Phyllites (7 ± 3)	Slates 7 ± 4

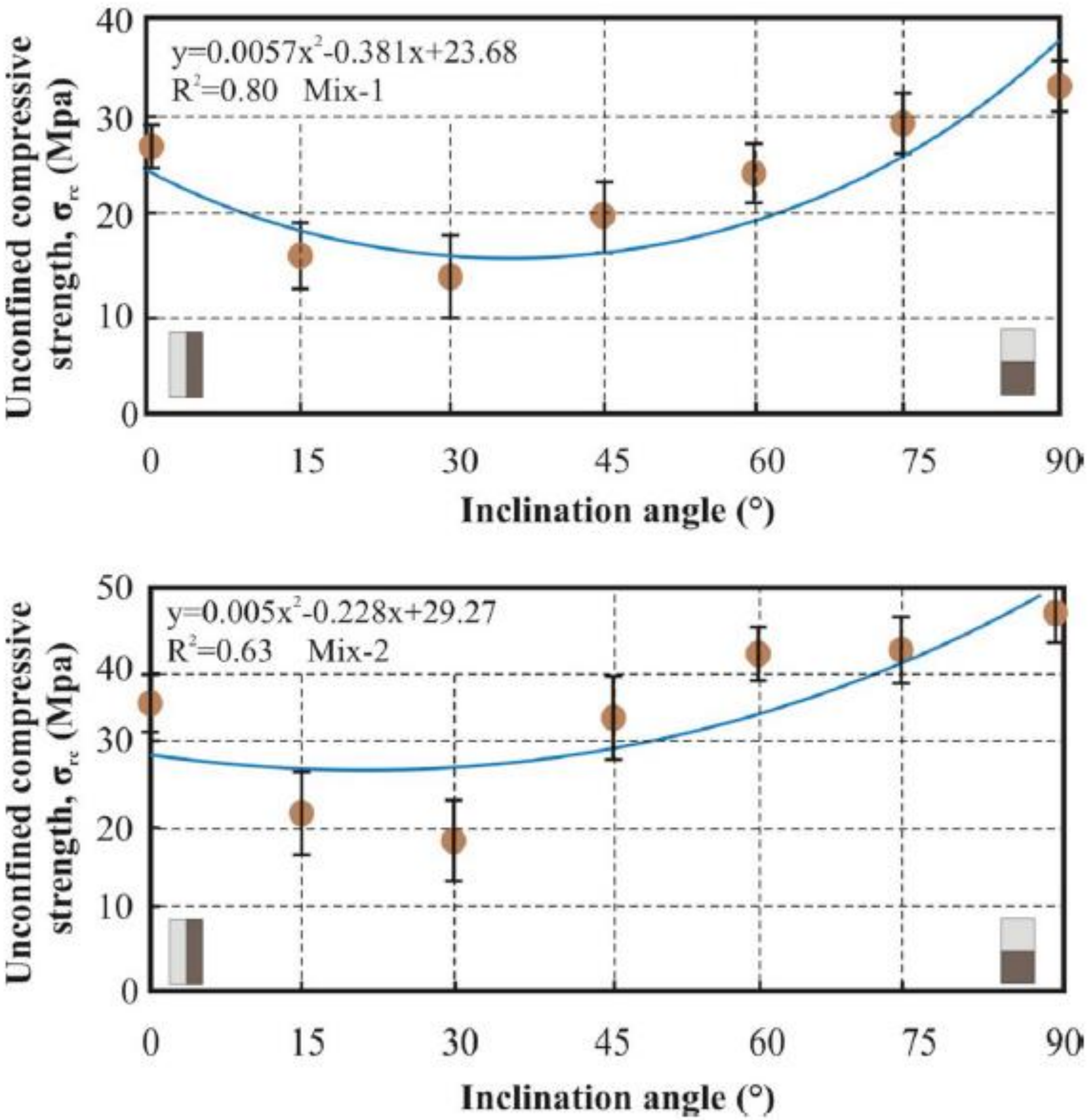
Values of the constant m_i for intact rock, by rock group (Hoek, 2007)

IGNEOUS	Plutonic	Light	Granite 32 ± 3 Granodiorite (29 ± 3)	Diorite 25 ± 5		
		Dark	Gabbro 27 ± 3 Norite 20 ± 5	Dolerite (16 ± 5)		
	Hypabyssal		Porphyries (20 ± 5)		Diabase (15 ± 5)	Peridotite (25 ± 5)
	Volcanic	Lava		Rhyolite (25 ± 5) Andesite 25 ± 5	Dacite (25 ± 3) Basalt (25 ± 5)	Obsidian (19 ± 3)
		Pyroclastic	Agglomerate (19 ± 3)	Breccia (19 ± 5)	Tuff (13 ± 5)	



Anisotropic and foliated rocks (rock-concrete)

[Selcuk, L. & Asma, D., 2019]

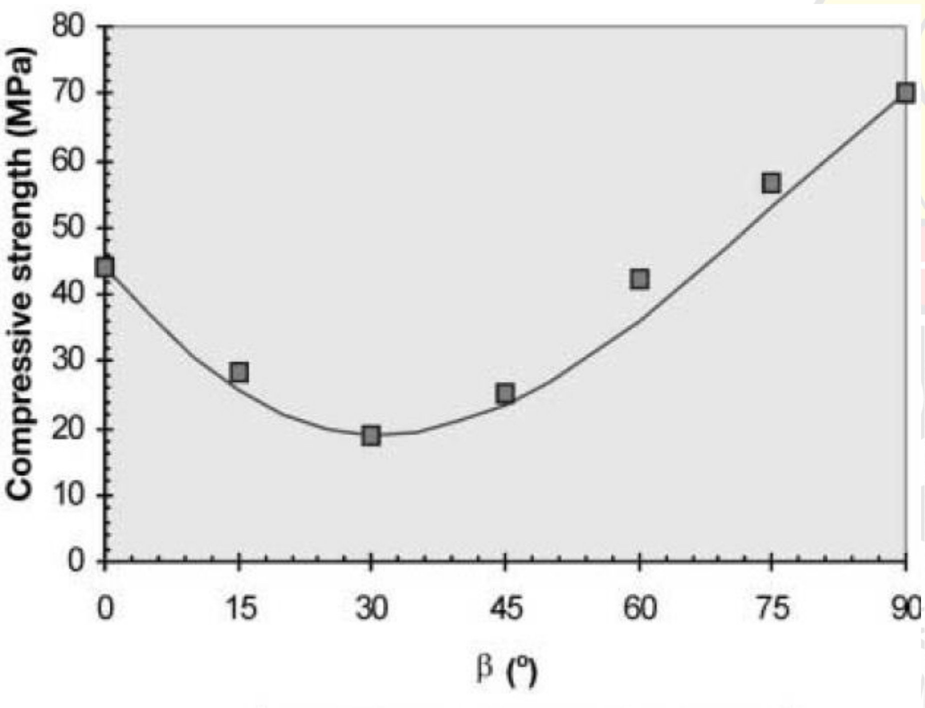


Anisotropic and foliated rocks (rock-concrete)

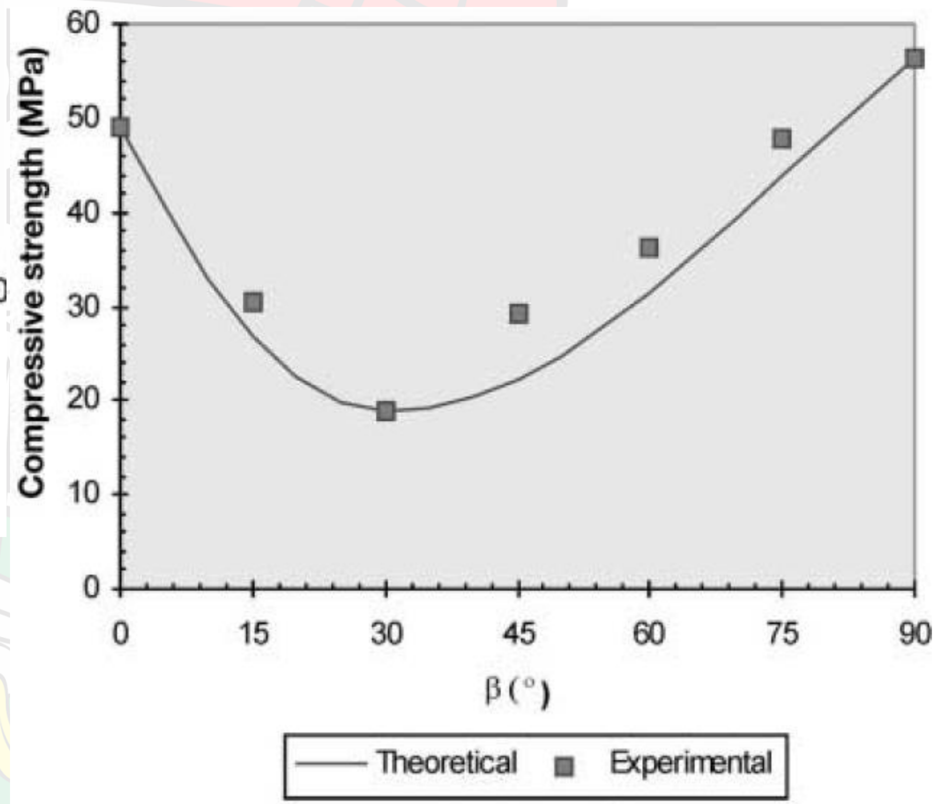
Selcuk, L. & Asma, D., 2019

Anisotropic and foliated rocks (mudrocks)

Ajalloeian, R. & Lashkaripour, G.R. 2000



(siltshale)



(mudshale)

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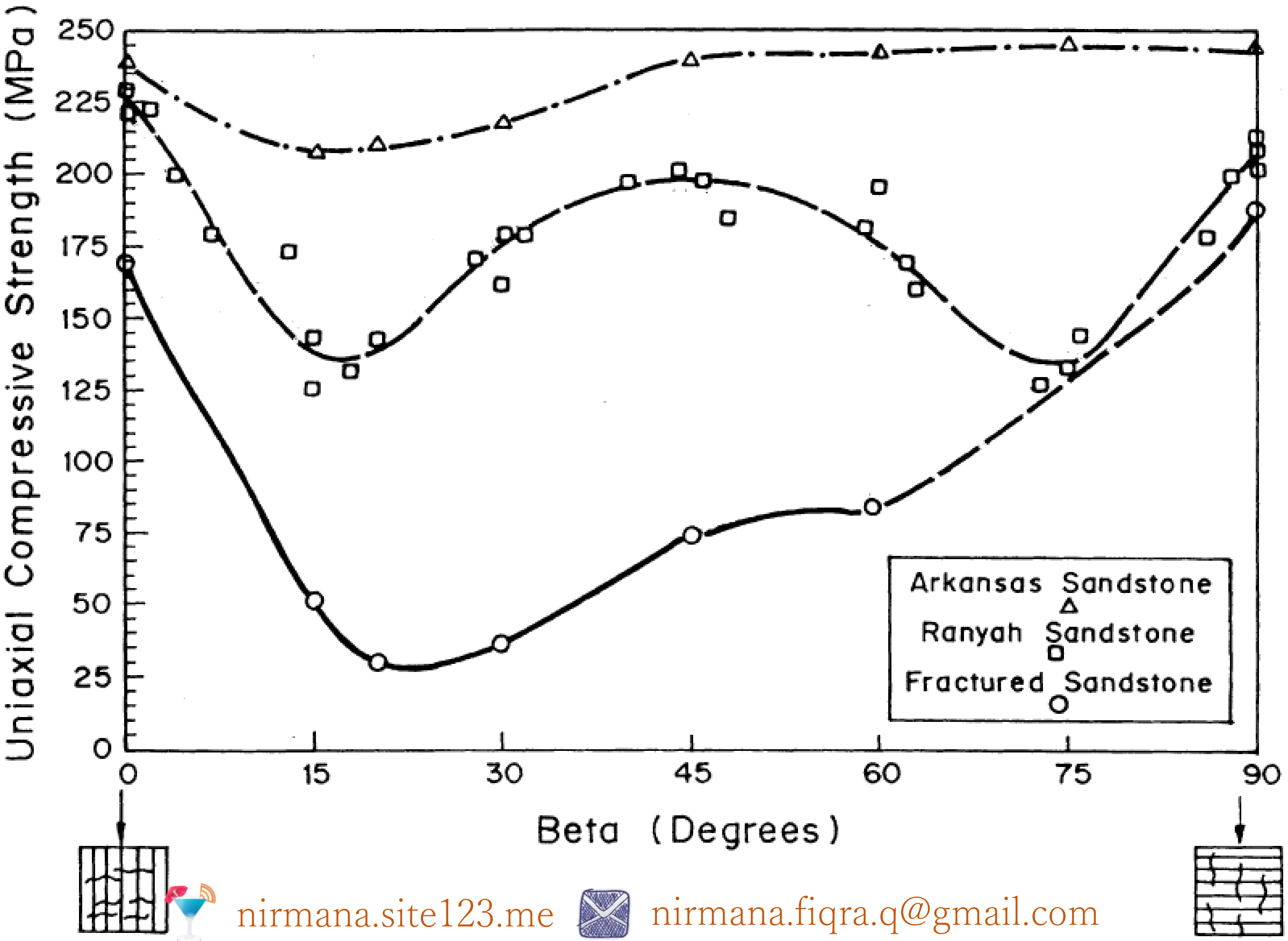
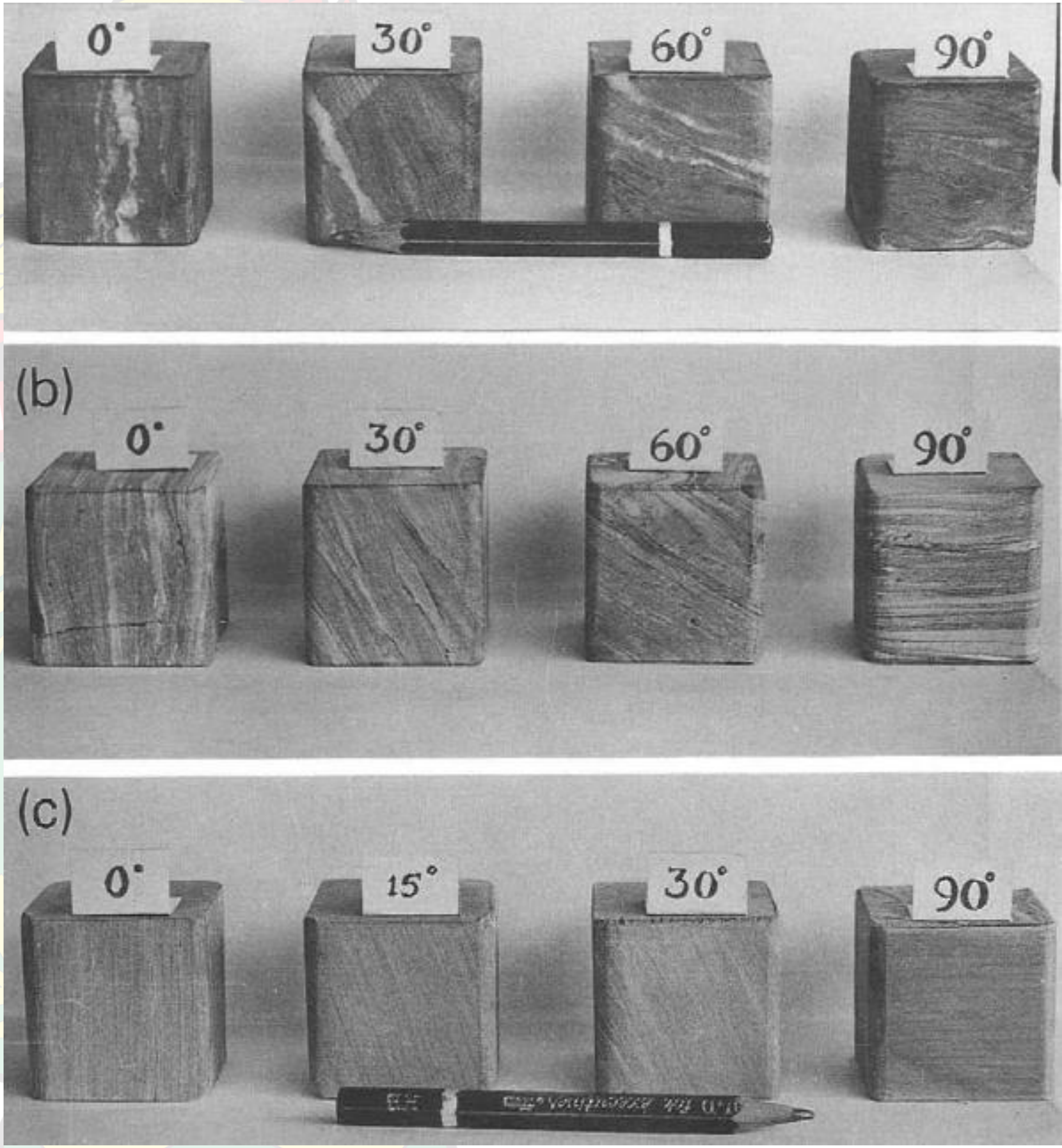
Anisotropic and foliated rocks (sandstone)

Al-Harthi, A.A. 1998



Anisotropic and foliated rocks (phyllite)
[Ramamurthy, T., Rao, G.V., and Singh, J. 1992]



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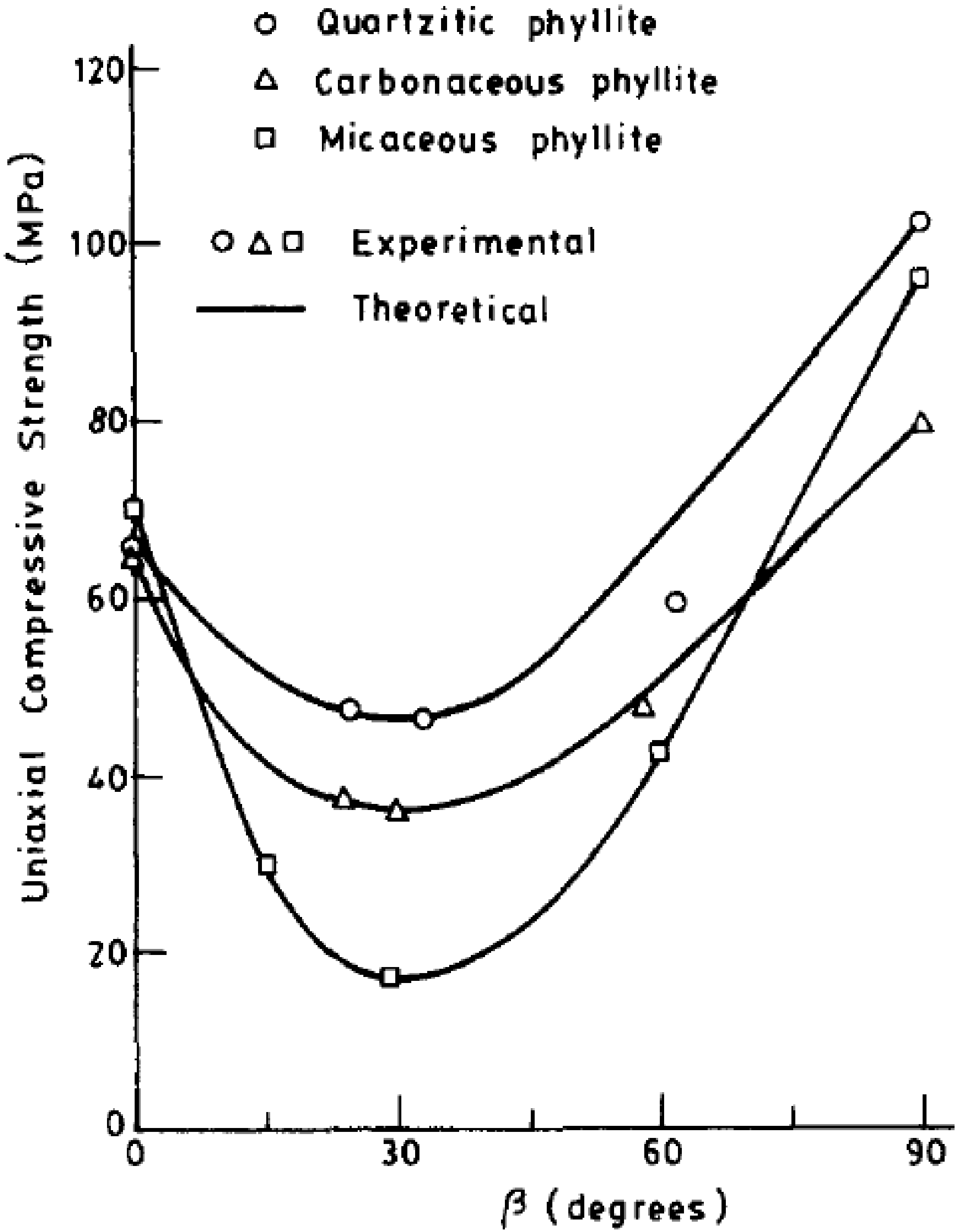

Anisotropic and foliated rocks (phyllite)

Ramamurthy, T., Rao, G.V., and Singh, J. 1992

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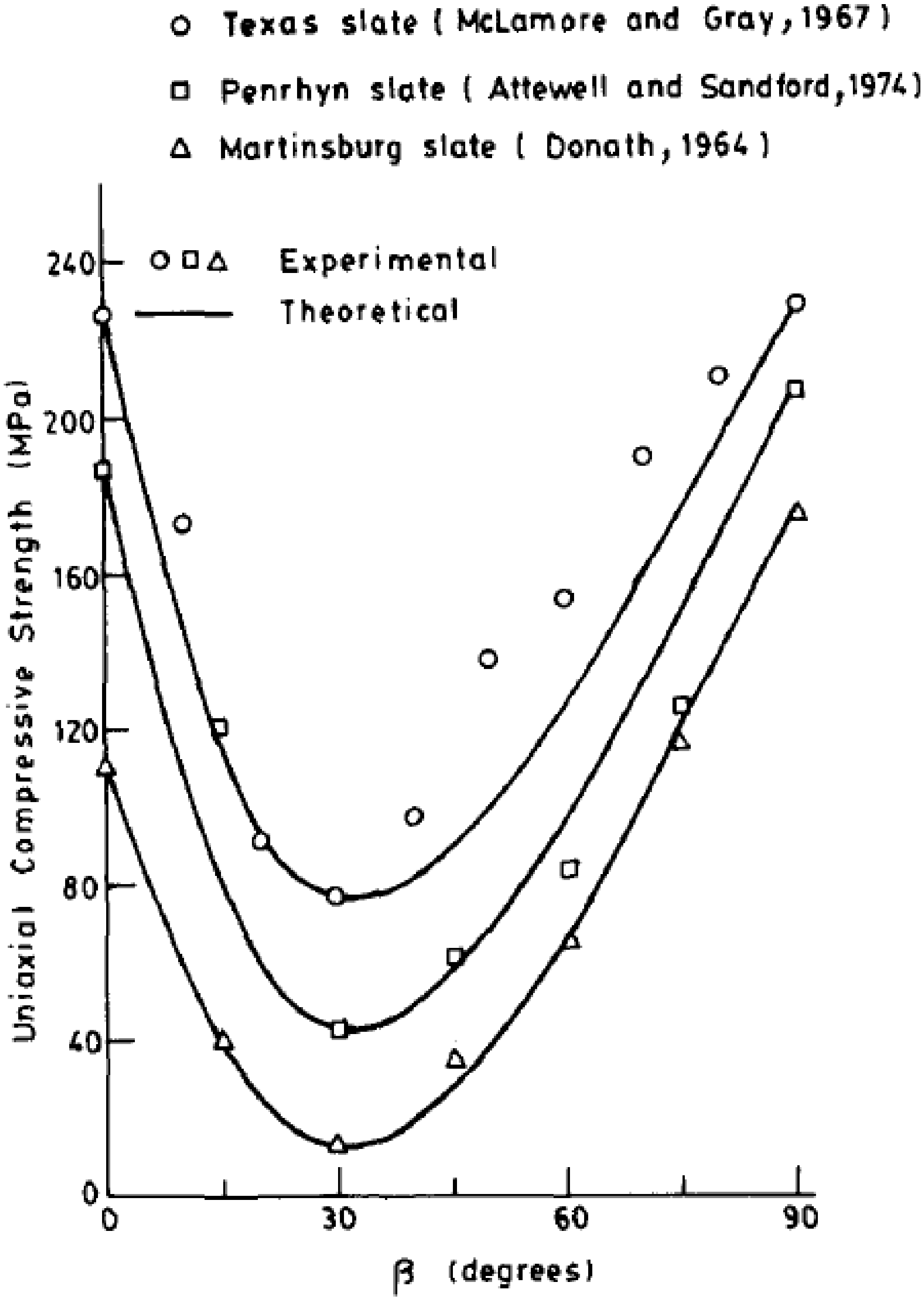

Anisotropic and foliated rocks (slate)

[Ramamurthy, T., Rao, G.V., and Singh, J. 1992]

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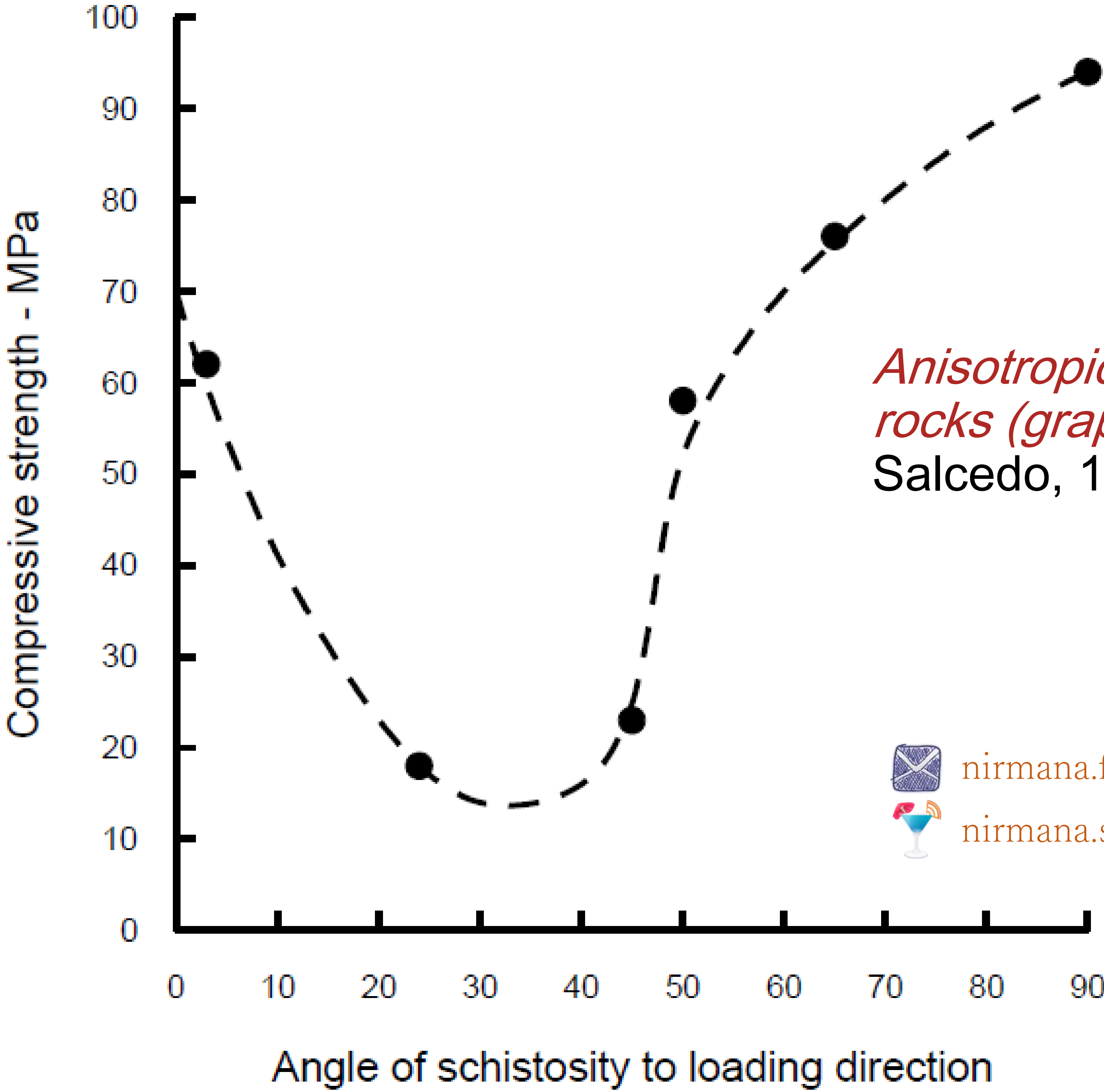

Anisotropic and foliated rocks (graphitic phyllite)

Salcedo, 1983

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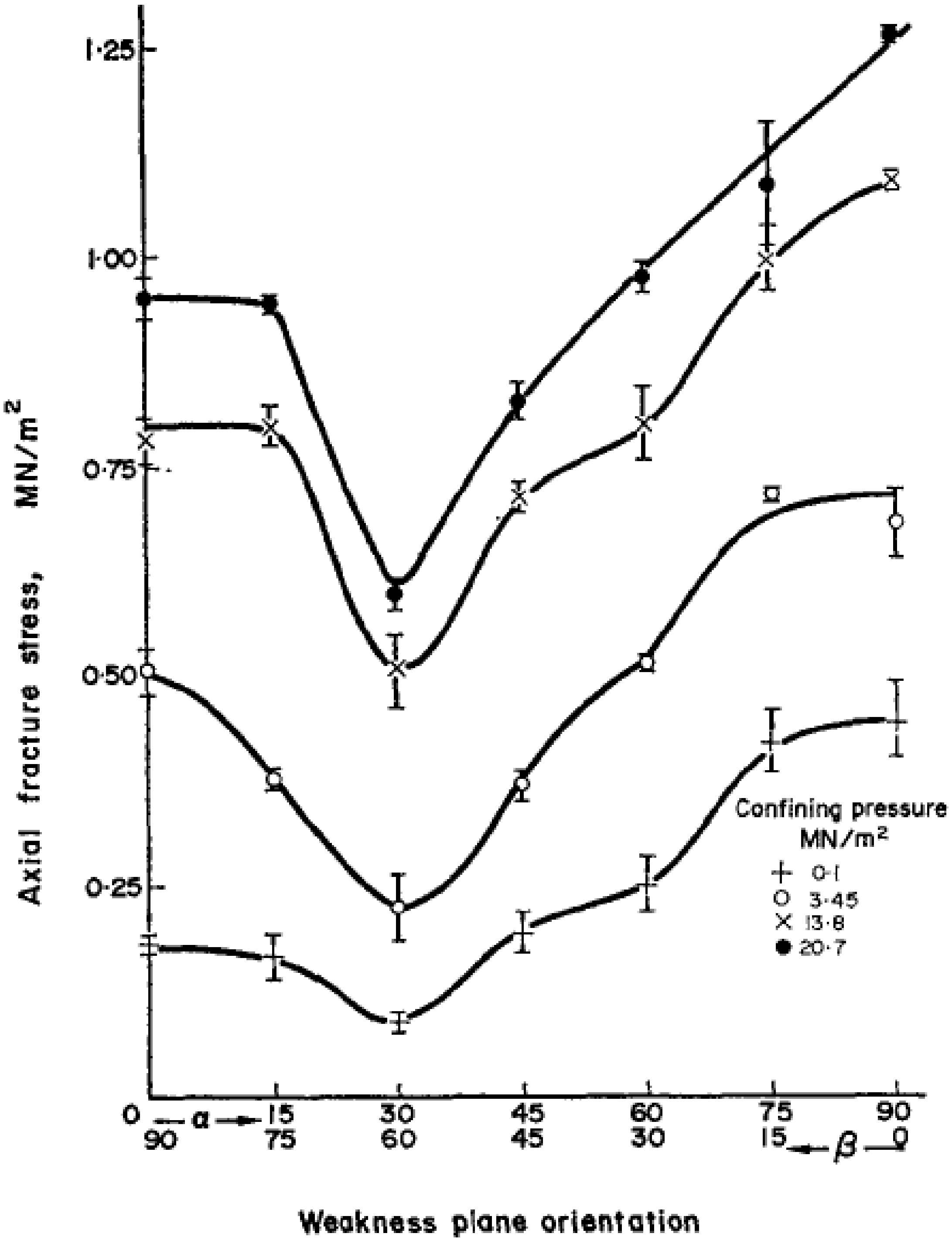

Anisotropic and foliated rocks (by triaxial compression)

[Pomeroy, C.D., Hobbs, D.W., and Mahmoud, A. 1971]

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Size Effect

It is well known that mechanical tests results in rock mechanics clearly show a scale effect. Increasing the size of specimen diminishes UCS, reducing the size increase UCS.

The effect of diameter changes (until 200 mm) can be assessed from Hoek & Brown formula, but there are no agreed formulations for bigger sizes.

Size Effect

Based upon an analysis of published data, Hoek and Brown (1980) have suggested that the UCS σ_{cd} of a rock specimen with a diameter of *d* mm is related to the UCS σ_{c50} of a 50 mm diameter sample by the following relationship:

$$\sigma_{cd} = \sigma_{c50}\left(\frac{50}{d}\right)^{0.18}$$

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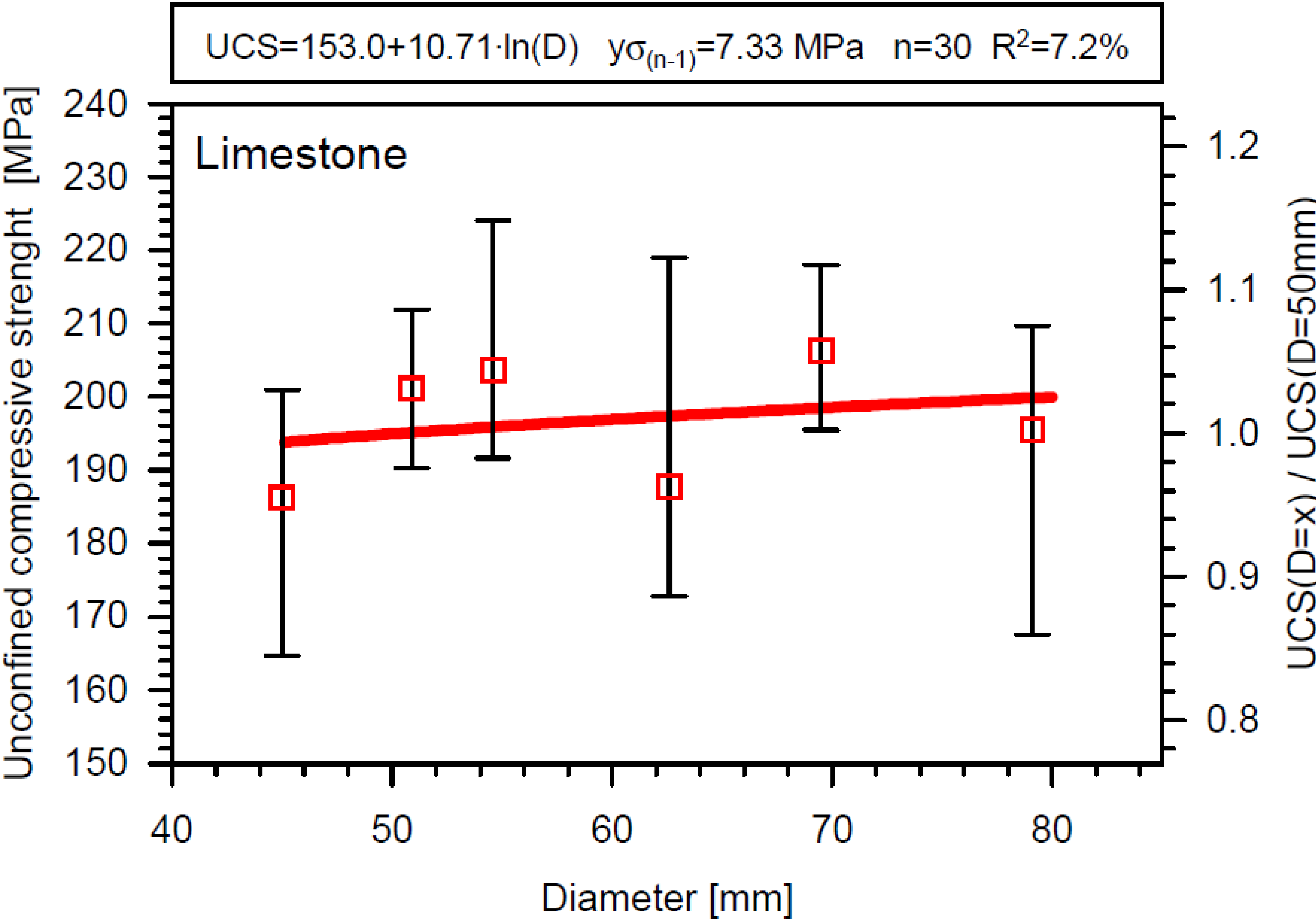
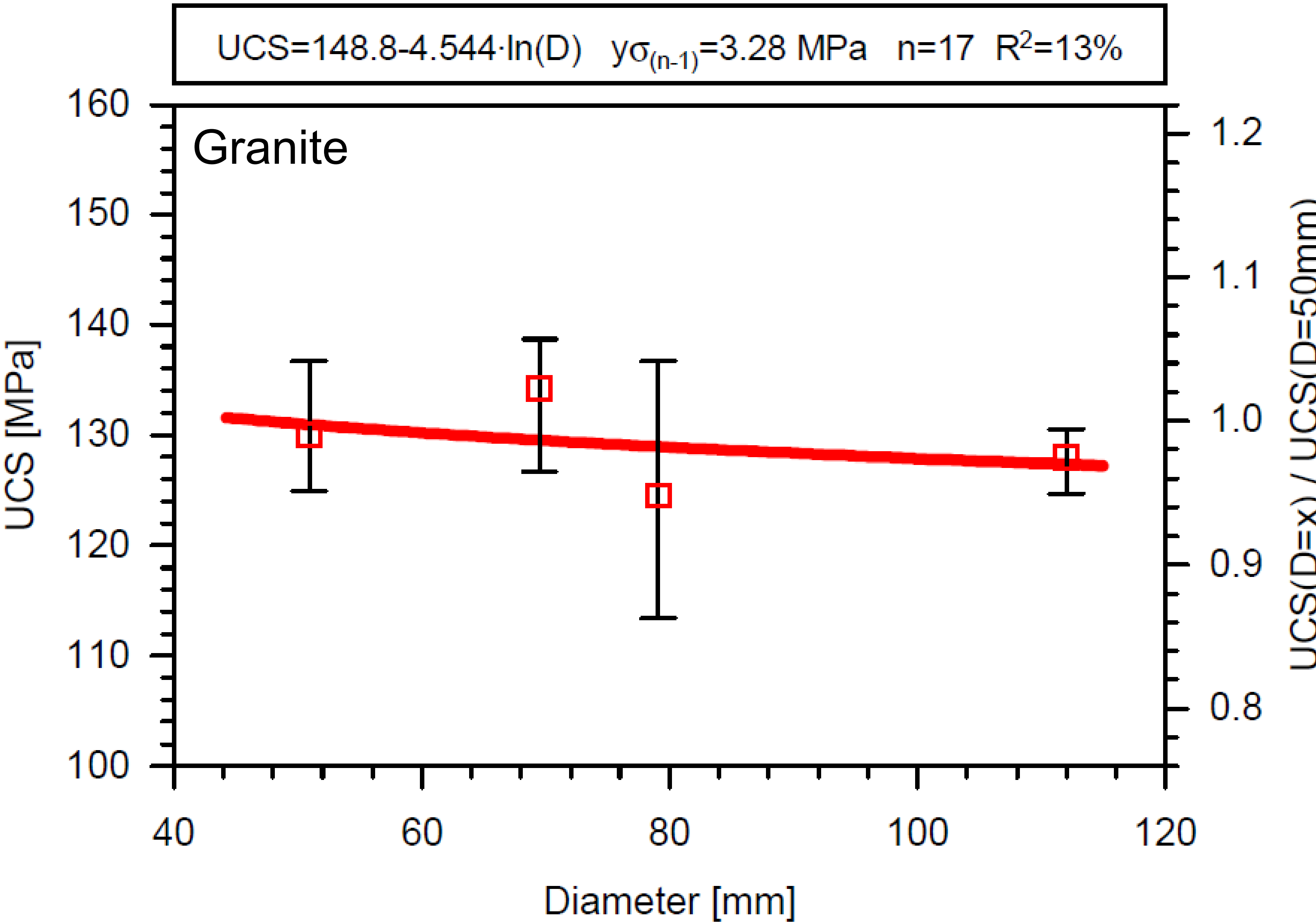
Size Effect

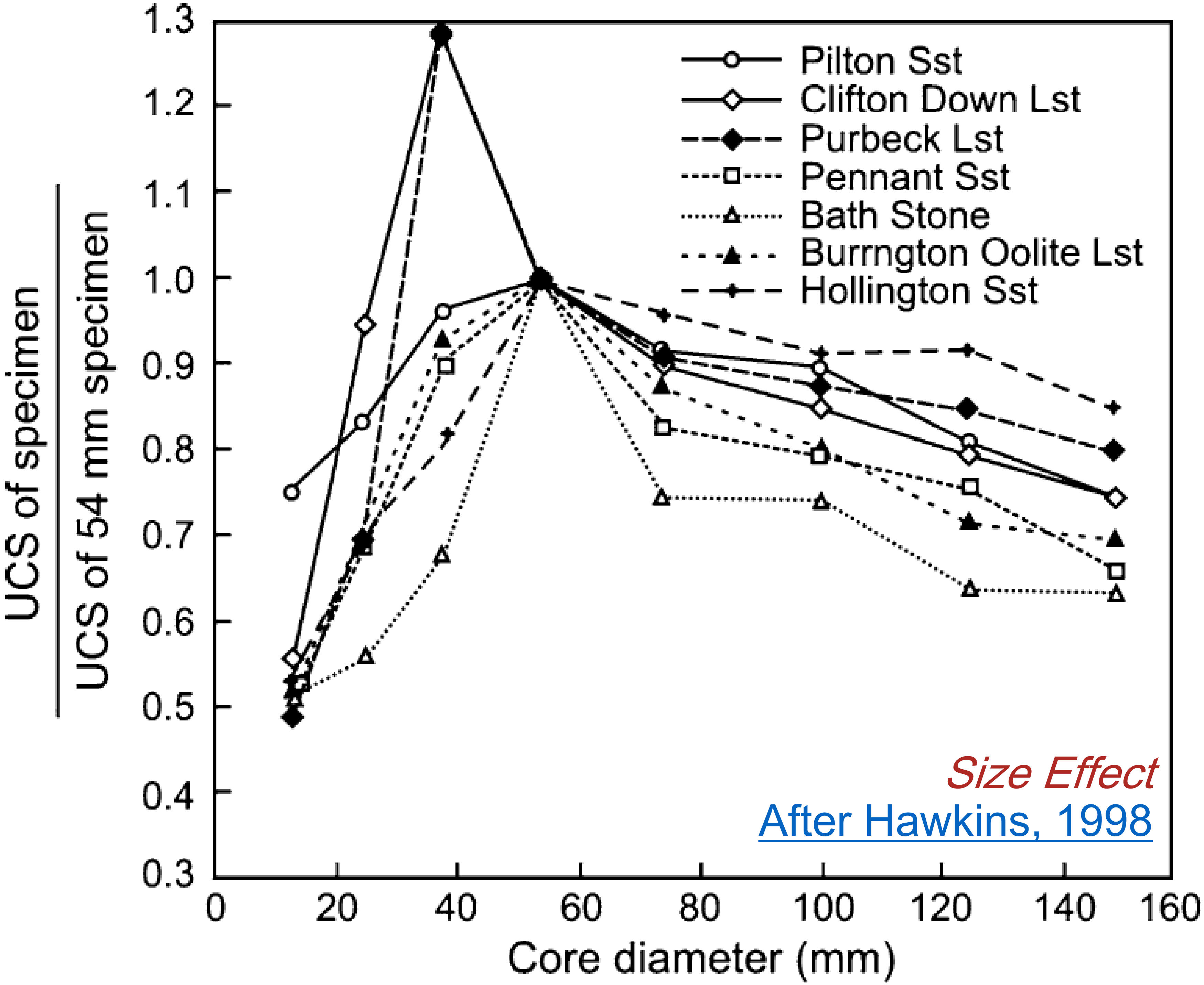
Thuro, K. 2001



Size Effect

Thuro, K. 2001

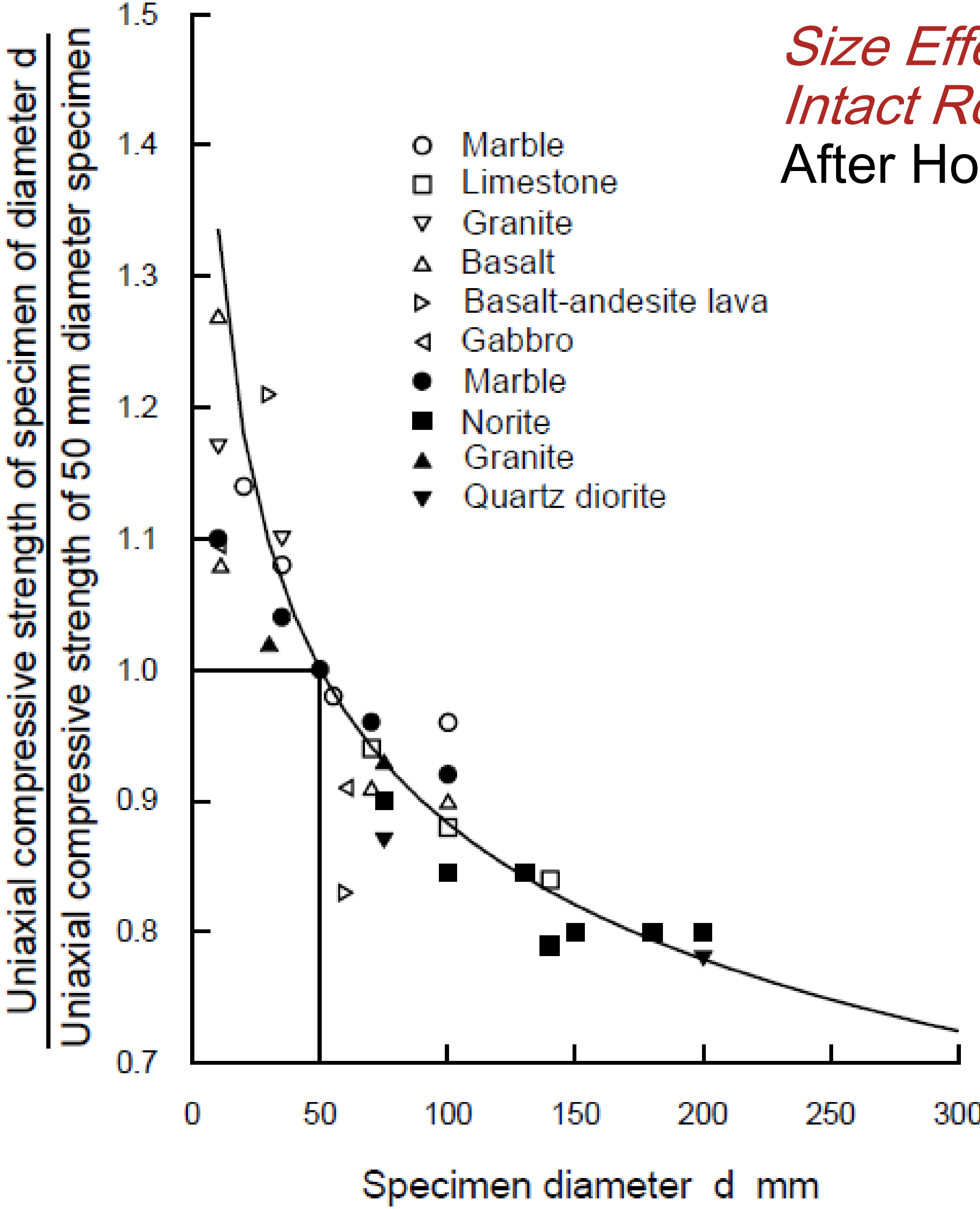




1.3
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
UCS of specimen
UCS of 54 mm specimen
0
20
40
60
80
100
120
140
160
Core diameter (mm)
Pilton Sst
Clifton Down Lst
Purbeck Lst
Pennant Sst
Bath Stone
Burrngton Oolite Lst
Hollington Sst
Size Effect
After Hawkins, 1998

Size Effect on The Strength of Intact Rock

After Hoek and Brown, 1980



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Size Effect

It is suggested that the reduction in strength is due to the greater opportunity for failure through and around grains, the 'building blocks' of the intact rock, as more and more of these grains are included in the test sample.

Eventually, when a sufficiently large number of grains are included in the sample, the strength reaches a constant value.

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Shape Effect

UCS is bigger in specimens with lower ratio. The suggested method proposes an aspect ratio of 2.5-3.0 Mogi (2007). Specimens with ratio<2.5 increased their UCS, relative increase was 10-15%. With bigger than 2.5 aspects ratio the strength diminishes in very small quantities.

Shape Effect

Obert and Duvall (1967) reported that the *L/D* ratio had a significant effect on the crushing strength which could be corrected by:

$$UCS_{corrected} = UCS/(0.778 + 0.222D/L)$$

Shape Effect

Protodyakonov (1969; Kahraman and Alber, 2006) suggested another equation to convert the UCS values to that of specimen with 2:1 *L/D* ratio

$$UCS_{corrected} = 8UCS/(7 + 2D/L)$$

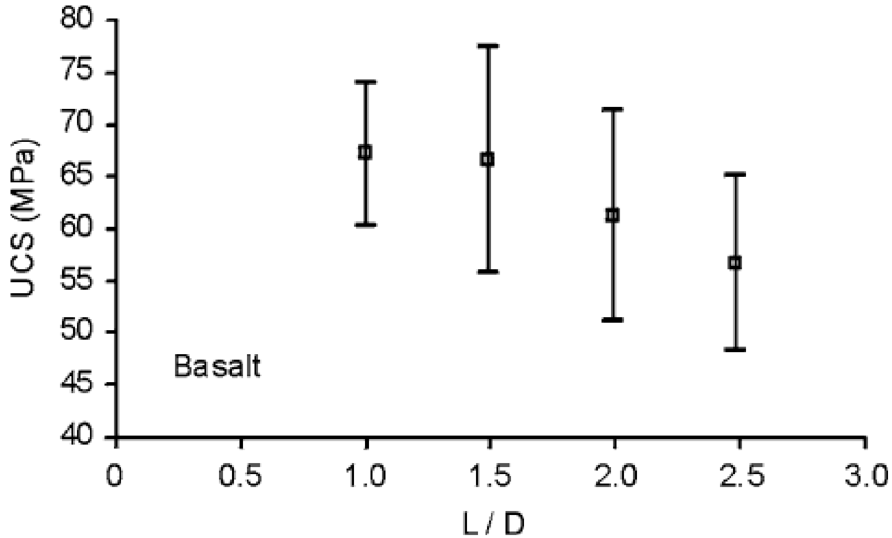
Shape Effect

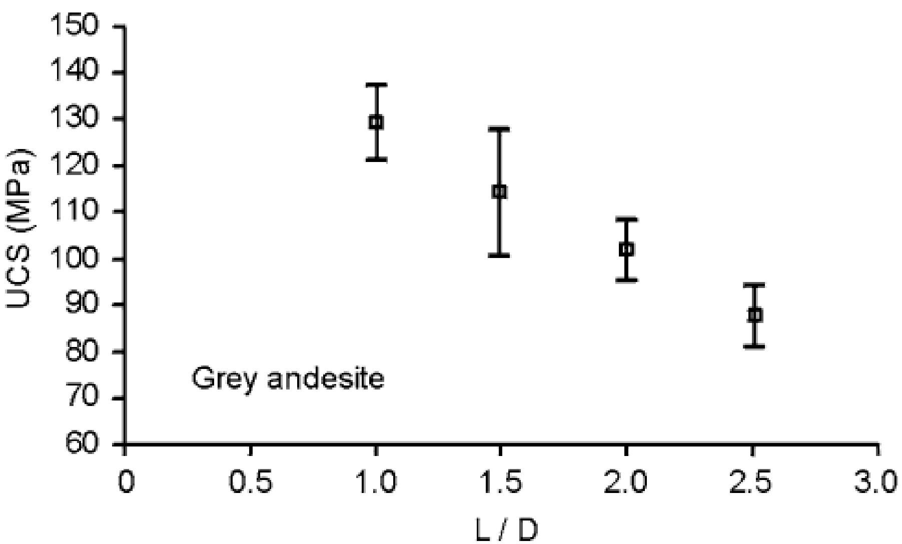
Tuncay, E. & Hasancebi, N. 2009

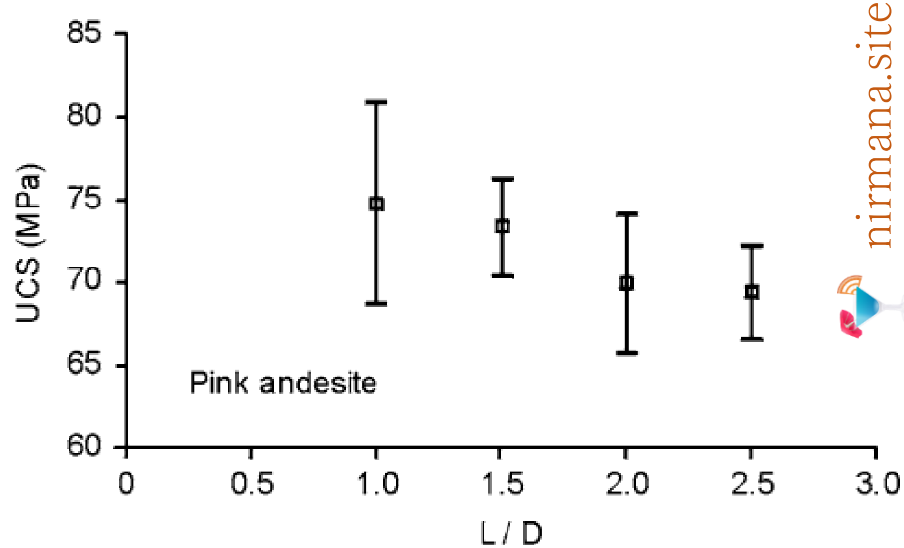
Rock type	Unit weight (kN/m^3)	*L/D* ratio	UCS (MPa)
Basalt	21.7	1.0	67.3 ± 6.9
		1.5	66.7 ± 10.8
		2.0	61.3 ± 10.1
		2.5	56.8 ± 8.8
Grey andesite	22.3	1.0	129.4 ± 8.0
		1.5	114.5 ± 13.5
		2.0	102.1 ± 6.5
		2.5	87.9 ± 6.4
Pink andesite	22.5	1.0	74.9 ± 6.0
		1.5	73.4 ± 3.0
		2.0	70.0 ± 4.2
		2.5	69.5 ± 2.8
Limestone	23.5	1.0	77.1 ± 2.1
		1.5	76.6 ± 1.5
		2.0	75.2 ± 1.8
		2.5	69.2 ± 3.1
Marble	26.4	1.0	87.1 ± 2.7
		1.5	85.7 ± 5.1
		2.0	84.4 ± 3.3
		2.5	82.0 ± 5.9
Siltstone	17.6	1.0	71.3 ± 1.7
		1.5	67.0 ± 0.4
		2.0	65.0 ± 1.4
		2.5	64.7 ± 1.7
Tuff	13.2	1.0	15.5 ± 1.5
		1.5	13.8 ± 0.5
		2.0	13.7 ± 0.3
		2.5	14.4 ± 0.4

Shape Effect

Tuncay, E. & Hasancebi, N. 2009

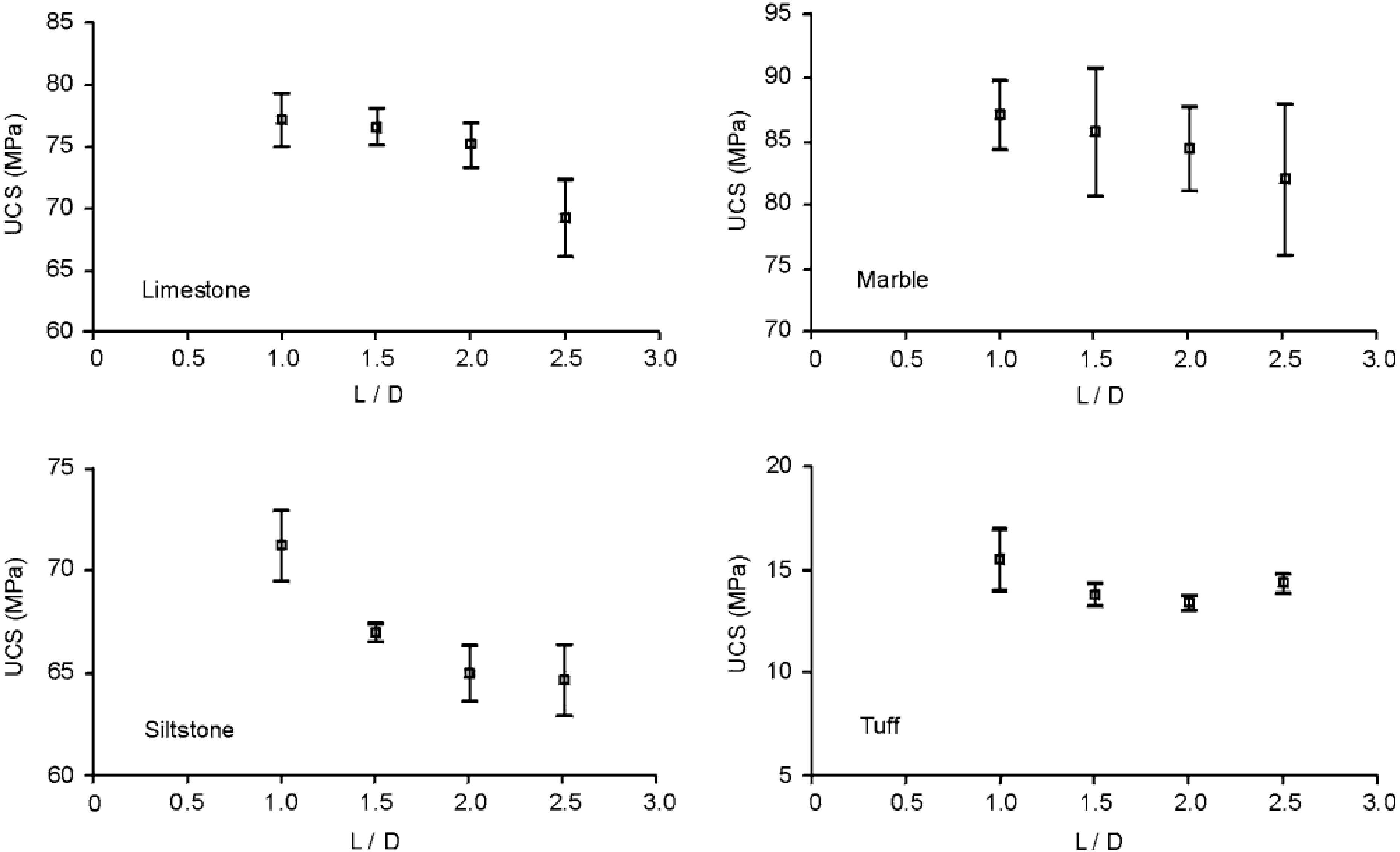






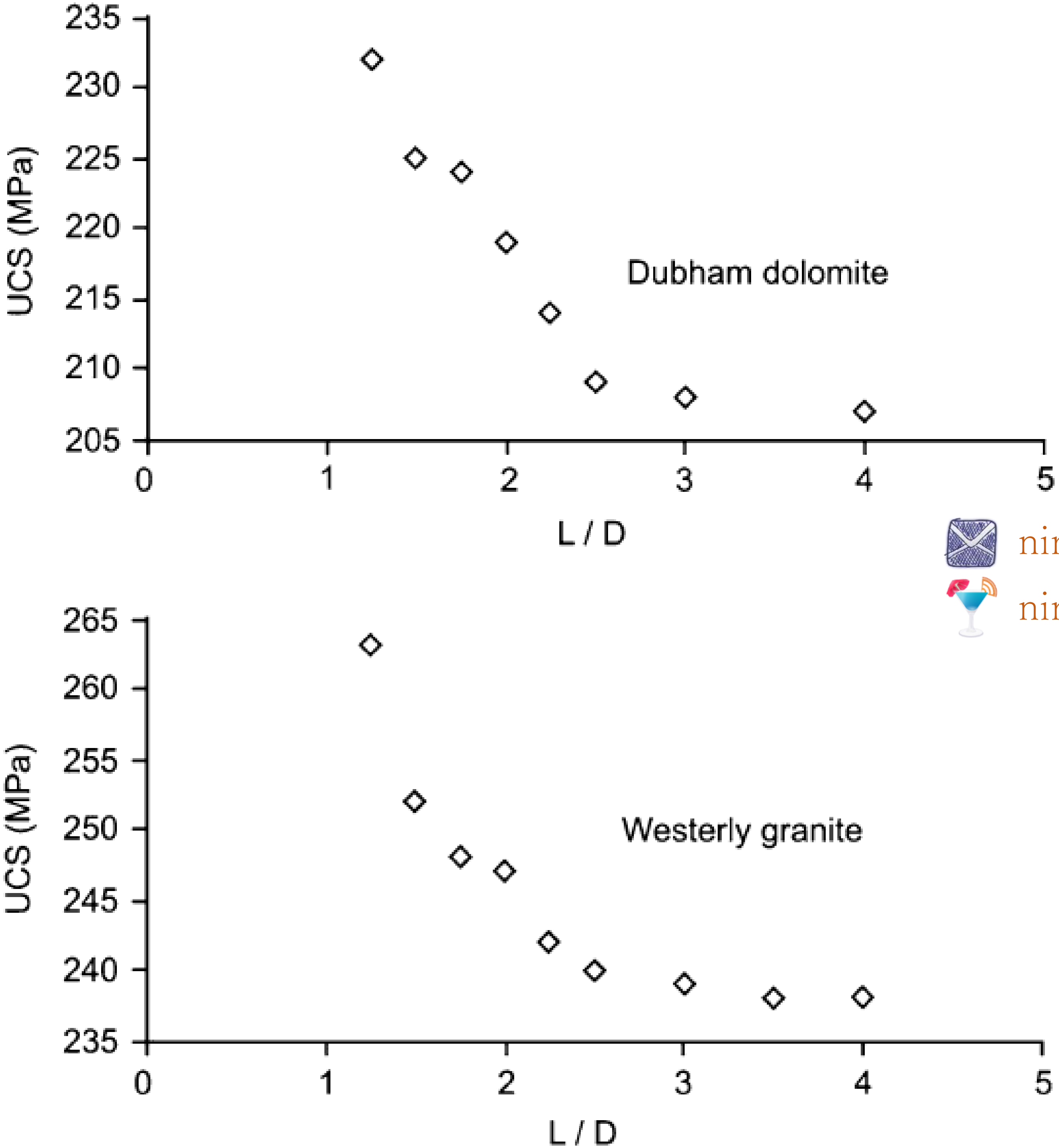
Shape Effect

Tuncay, E. & Hasancebi, N. 2009



Shape Effect

Mogi (1966, 2007)



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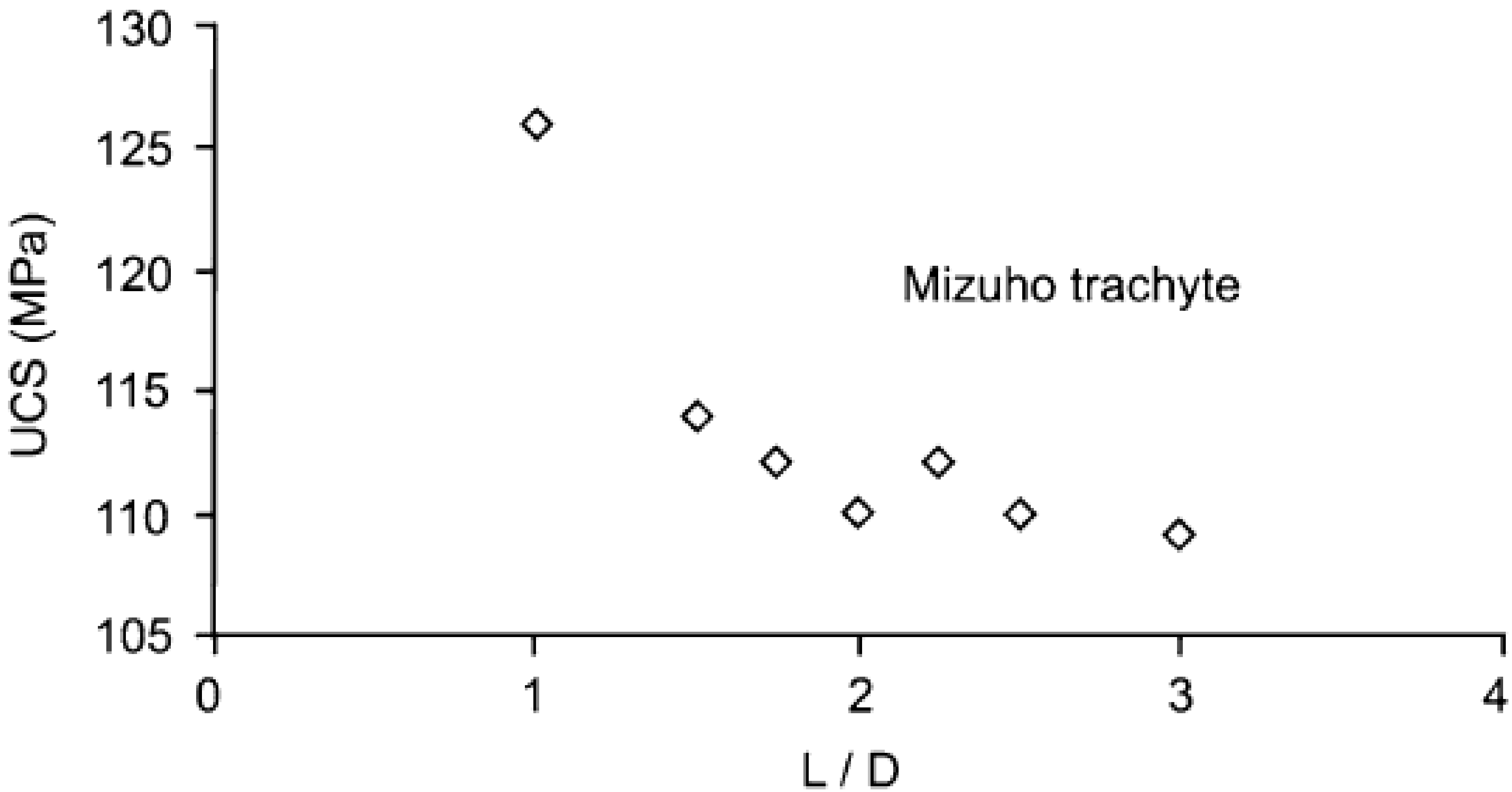
Shape Effect

Mogi (1966, 2007)



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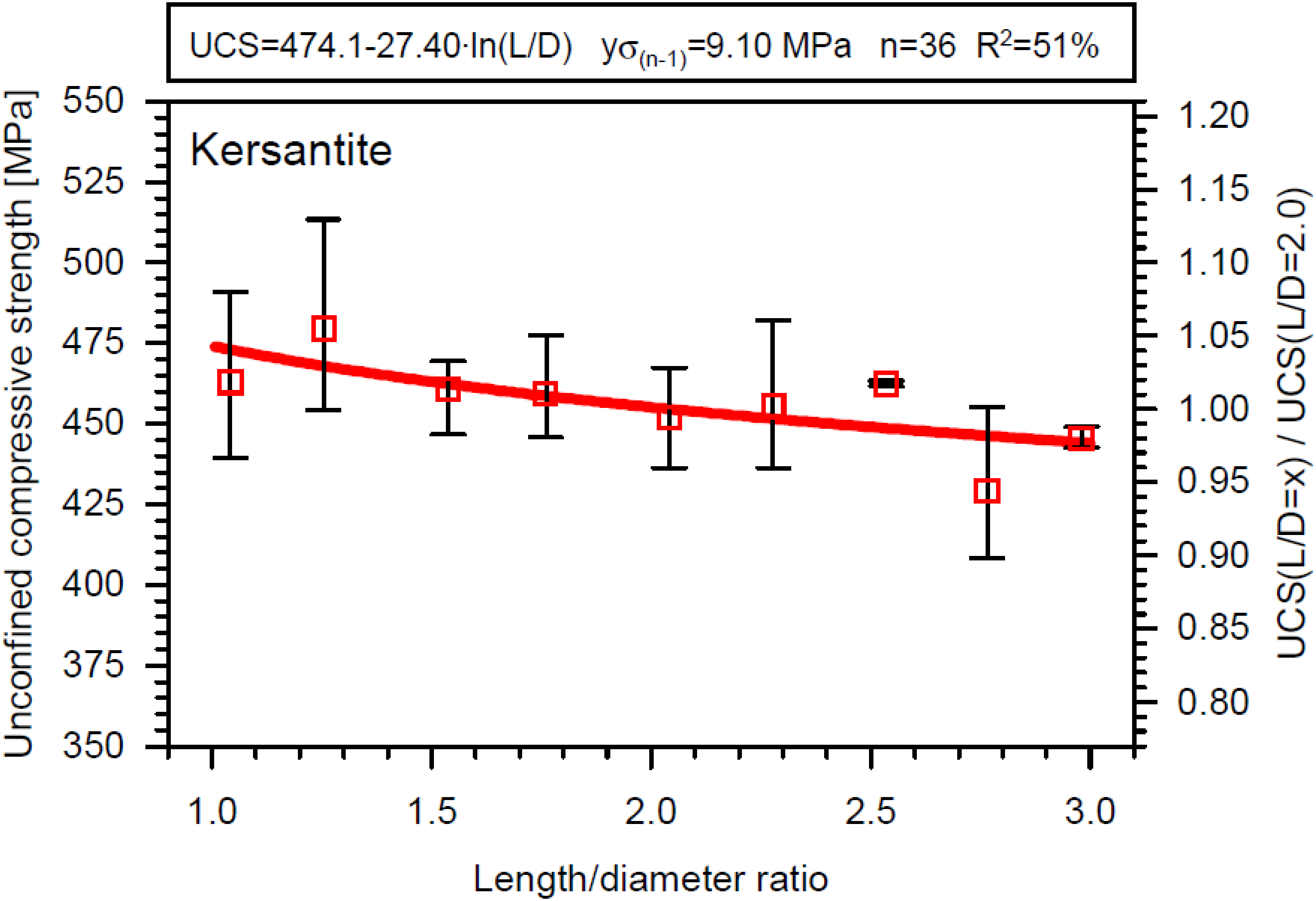
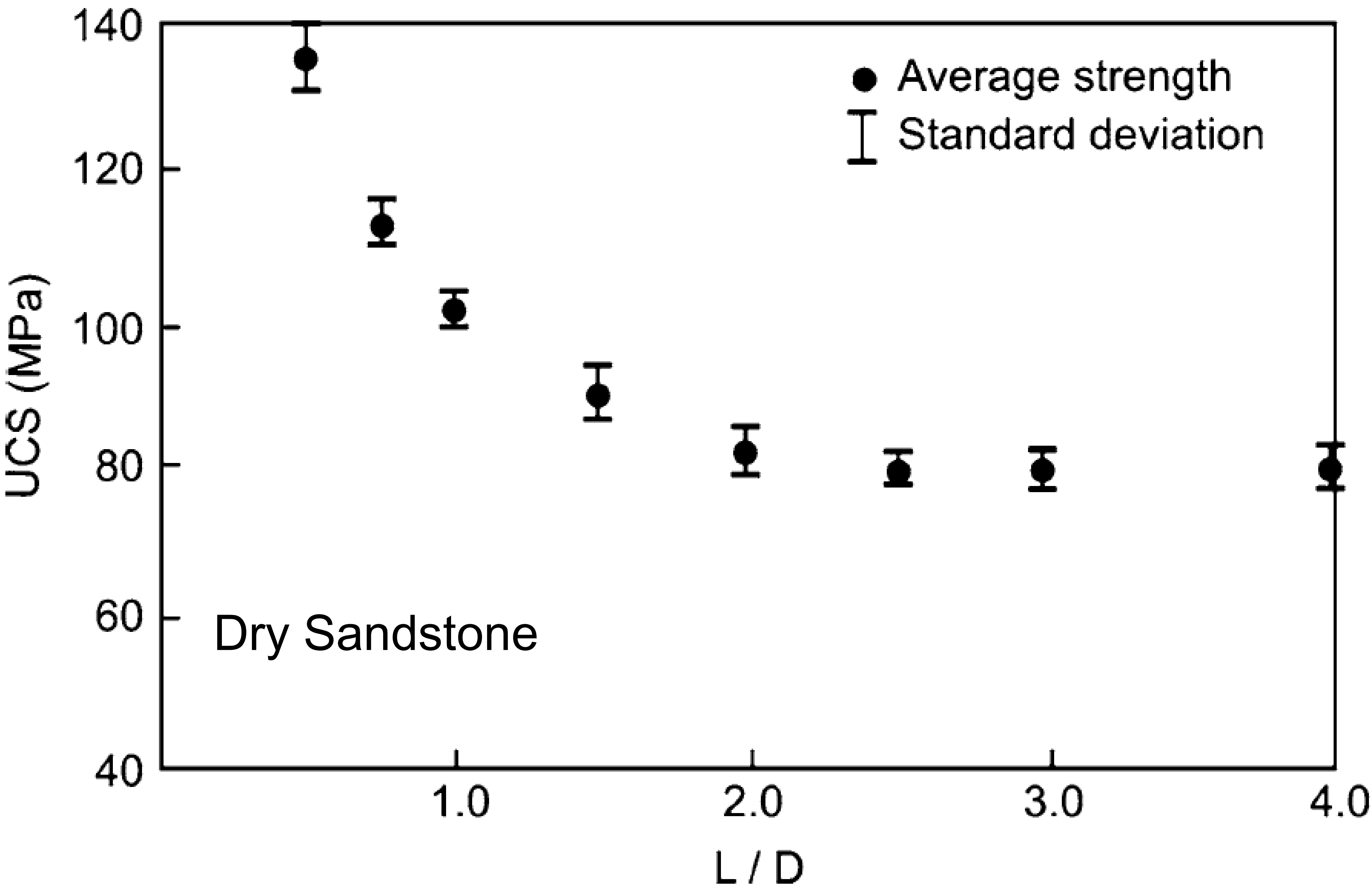
Shape Effect

Thuro, K. 2001



Shape Effect

John 1972 from Hawkins 1998



Shape Effect

Mogi, K. 1966

Rock	L/D	No. of Specimens	Apparent strength, MPa	Relative strength, %
Dunham dolomite	1.25	2	232 ± 1	111.5
	1.50	3	225 ± 3	107.5
	1.75	2	224 ± 3	107
	2.00	3	219 ± 3	104.5
	2.25	2	214 ± 0	102.5
	2.50	2	209 ± 0	100
	3.00	2	208 ± 0	99.5
	4.00	1	207	99
Westerly granite	1.25	2	263 ± 9	109.5
	1.50	2	252 ± 2	105
	1.75	2	248 ± 2	103.5
	2.00	3	247 ± 5	102.5
	2.25	3	242 ± 4	100.5
	2.50	4	240 ± 6	100
	3.00	2	239 ± 4	99.5
	3.50	2	238 ± 6	99
	4.00	3	238 ± 5	99
Mizuho trachyte	1.00	1	126	115.5
	1.50	1	114	104.5
	1.75	1	112	103
	2.00	1	110	101
	2.25	1	112	102.5
	2.50	1	110	100
	3.00	1	109	99.5

Form Effect

In many cases the rock samples are small and irregular (for instance when sampling at a tunnel face) and for simplicity cubic specimens are arranged for testing.

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Form Effect

This produce a double bias:

- ❑ the bigger samples from excavation will be the stronger ones;
- ❑ the cubic specimen has bigger UCS than a cylindrical one with ratio = 1.

(Romana, 2012)

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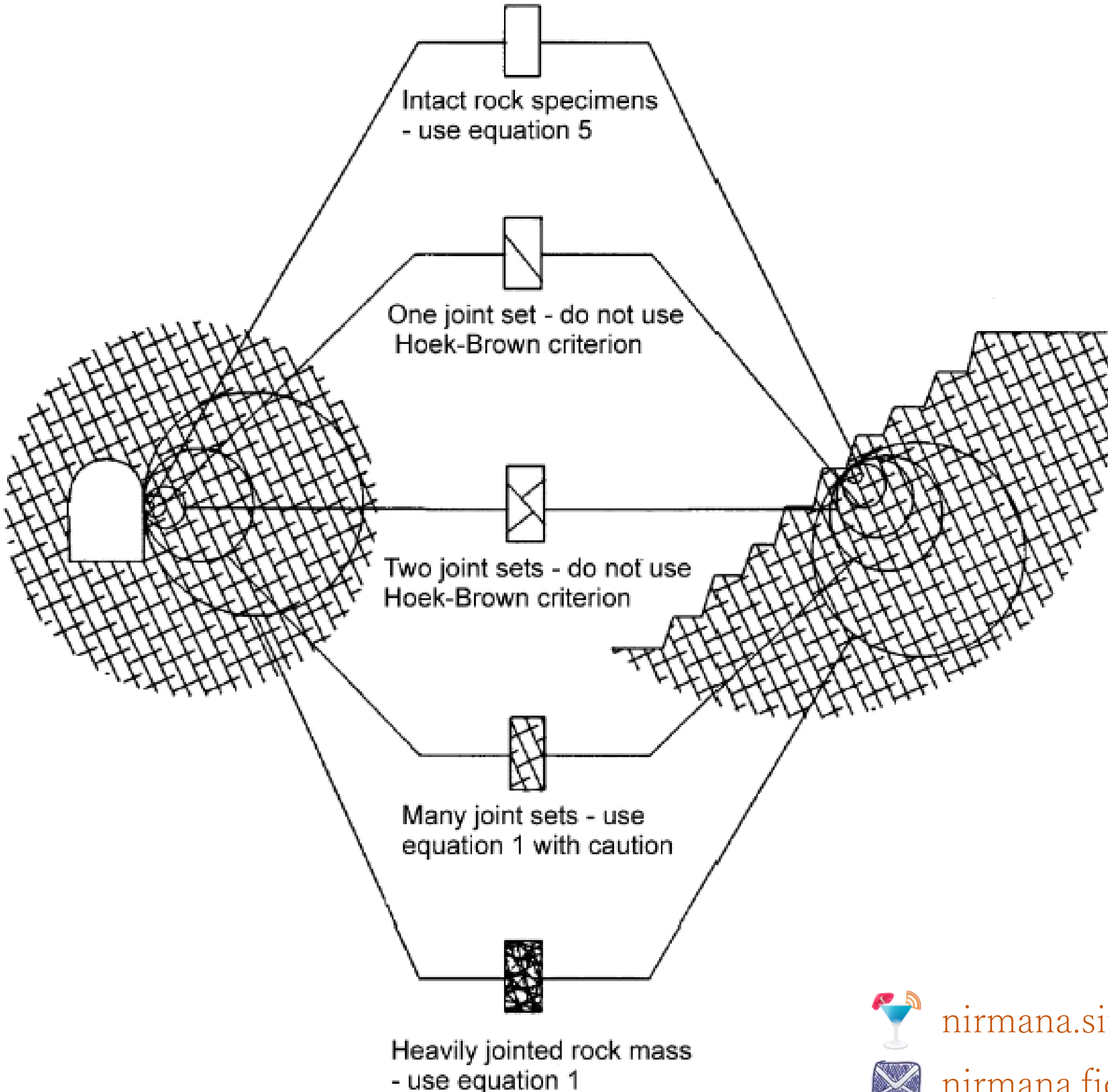
Form Effect

That increase is well known in concrete testing, British Standards Institution, 1992. for stresses less than 50 MPa it is very frequent that concrete cylindrical UCS be multiplied by 1.25, to get the equivalent cubical UCS.

Hoek, 2007



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Geological Strength Index (GSI)

The strength of a jointed rock mass depends on:

✓ the properties of the intact rock pieces; and

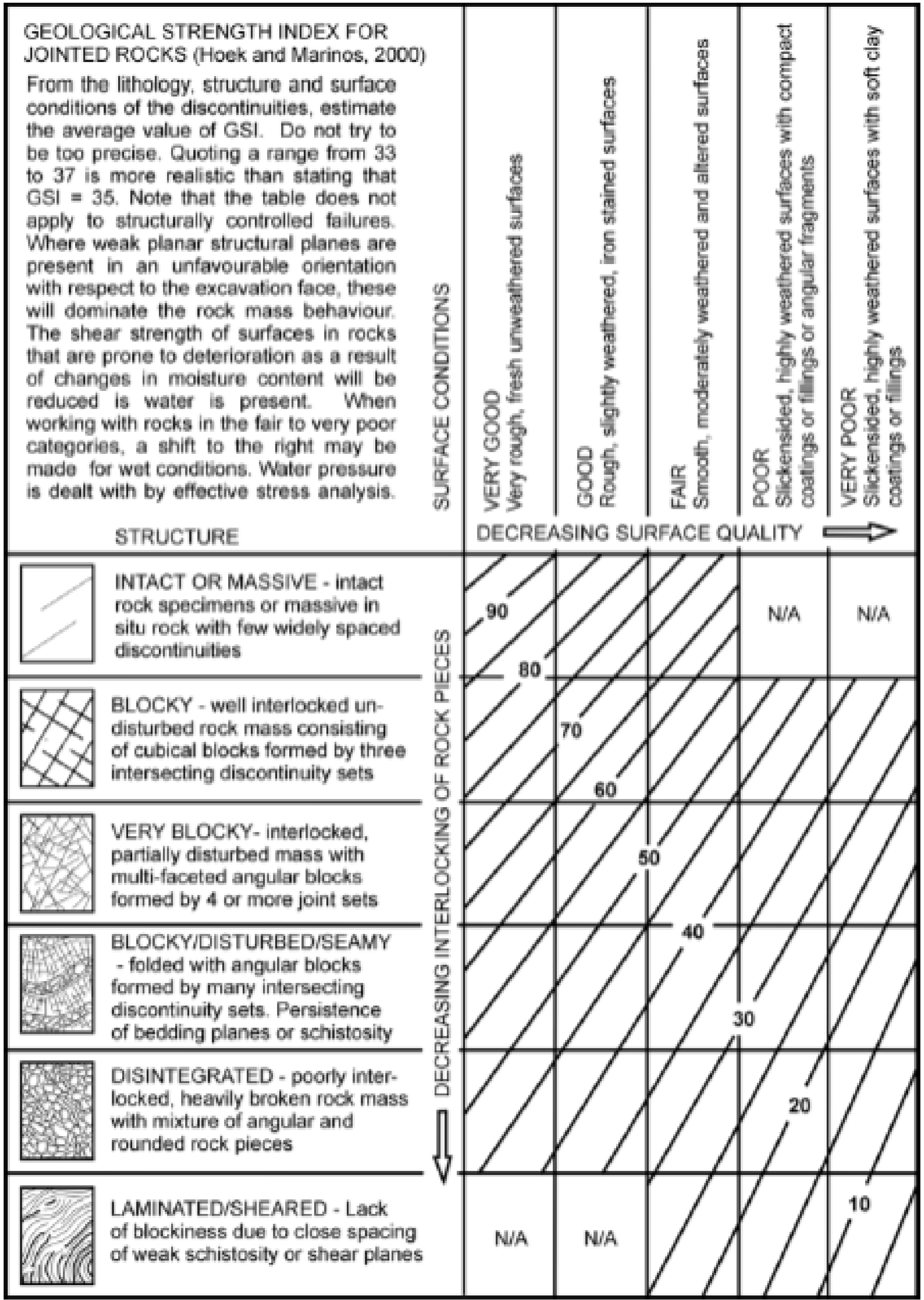
✓ discontinuity conditions

Geological Strength Index (GSI)

GSI, introduced by Hoek (1994) and Hoek, Kaiser, and Bawden (1995), provides a number which, when combined with the intact rock properties, can be used for estimating the reduction in rock mass strength for different geological conditions.

Blocky Rock Masses

Hoek, 2007



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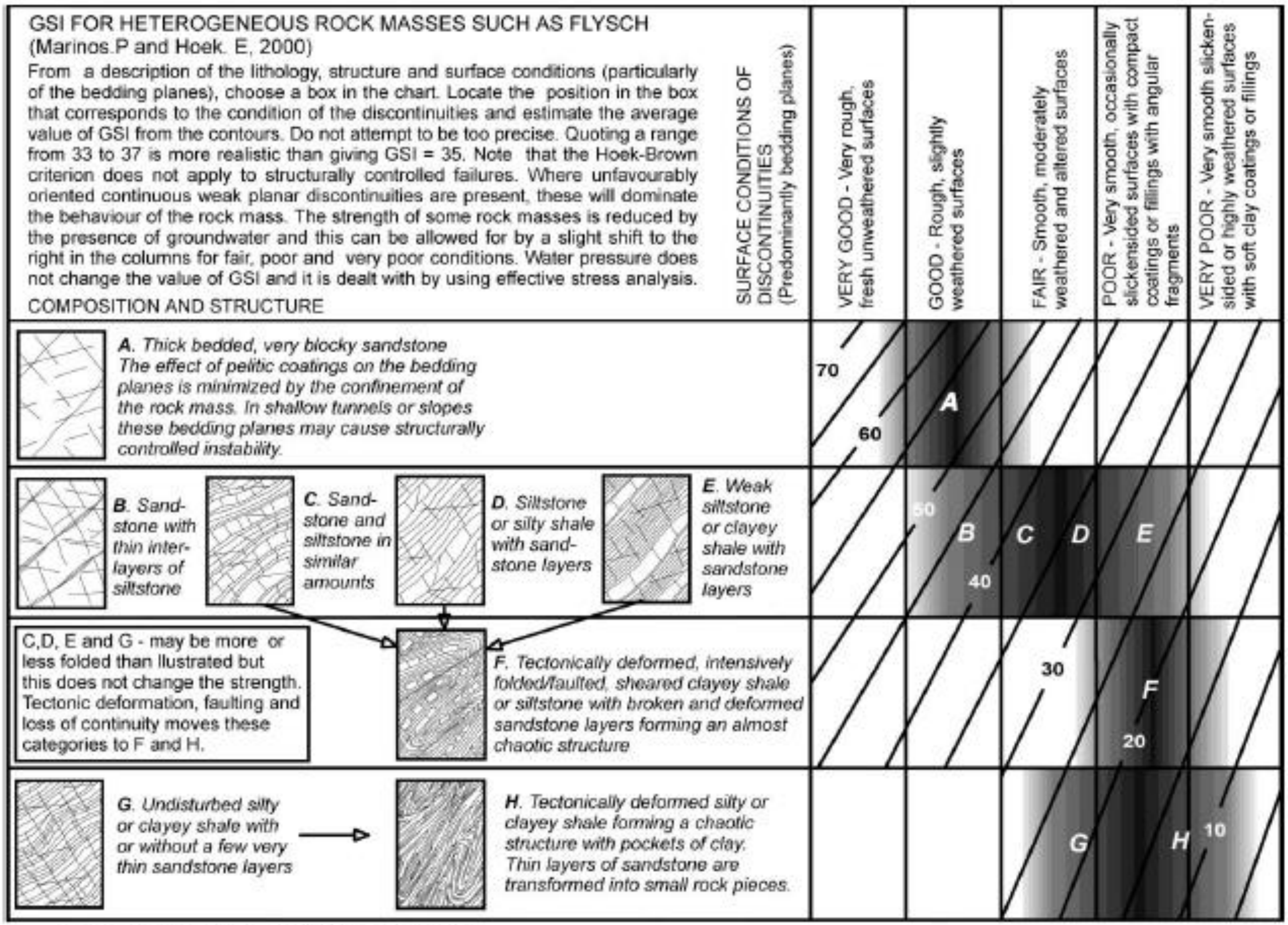
Heterogeneous Rock Masses (Hoek, 2007)



→ : Means deformation after tectonic disturbance

Geological Strength Index (GSI)

During the early years of the application of the GSI system, the value of GSI was estimated directly from RMR. *However*, this correlation has proved to be unreliable, particularly for poor quality rock masses and for rocks with lithological peculiarities that cannot be accommodated in the RMR classification. Consequently, it is recommended that **GSI should be estimated directly by means of the previous charts** and not from the RMR classification.

Whether borehole cores can be used to estimate the GSI value?

Borehole cores are the best source of data at depth but it has to be recognized that <u>it is necessary to extrapolate the one dimensional information provided by core to the three-dimensional rock mass</u>. However, this is a common problem in borehole investigation and most experienced engineering geologists are comfortable with this extrapolation process. <u>Multiple boreholes and inclined boreholes</u> are of great help the interpretation of rock mass characteristics at depth.

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Geological Strength Index (GSI)

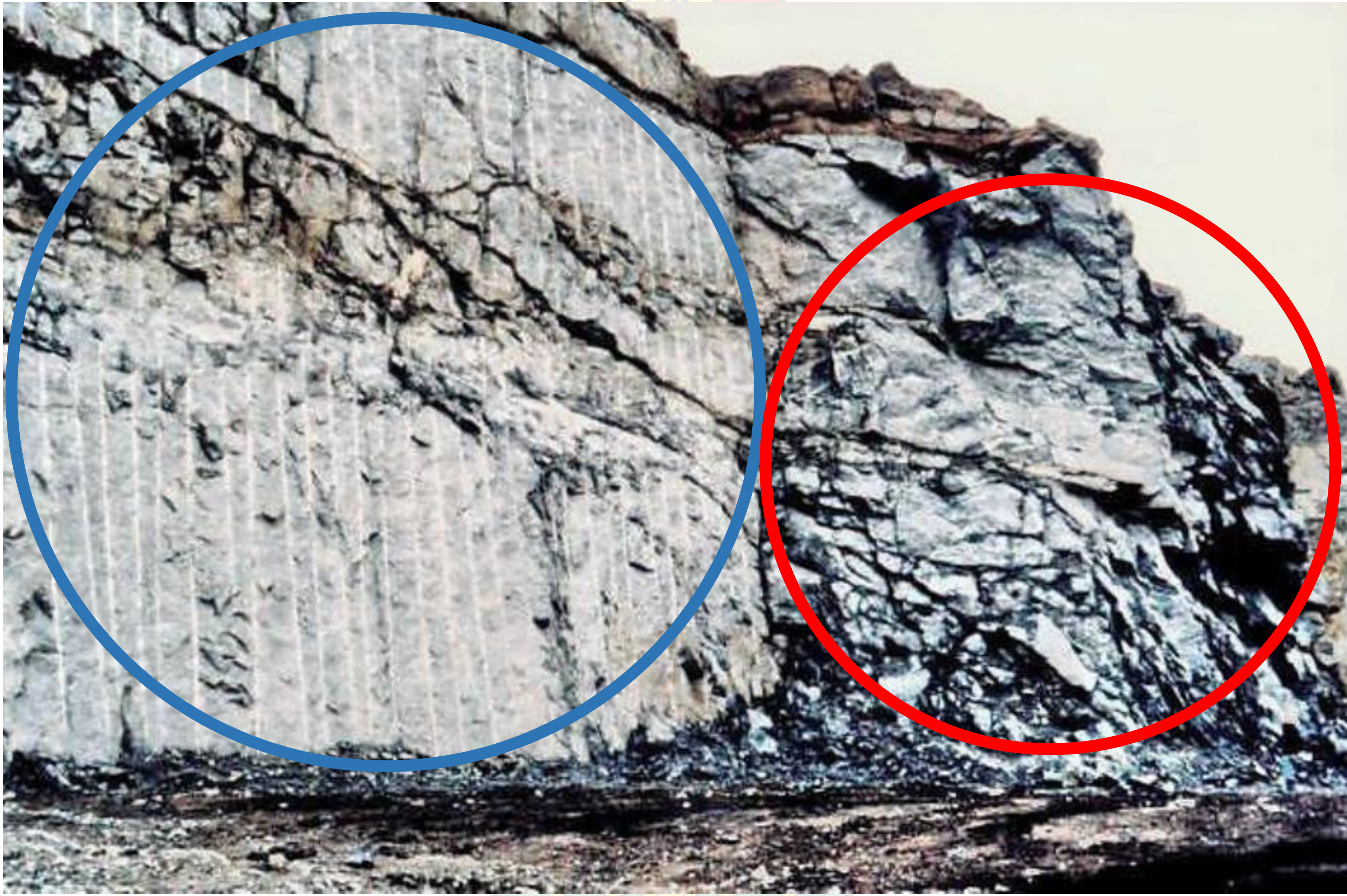
The most important decision to be made in using the GSI system is *whether or not it should be used*. If **the discontinuity spacing is large** compared with the dimensions of the tunnel or slope under consideration, the GSI tables and the Hoek-Brown criterion should not be used and the discontinuities should be treated individually.

Geological Strength Index (GSI)

Where **the discontinuity spacing is small** compared with the size of the structure, then the GSI tables can be used with confidence.

Geological Strength Index (GSI)

One of the practical problems that arises when assessing the value of GSI in the field is related to **blast damage**. Wherever possible, the undamaged face should be used to estimate the value of GSI since the overall aim is to determine the properties of the undisturbed rock mass.



Generalized Hoek-Brown Criterion

The Generalized Hoek-Brown failure criterion for **jointed rock masses** is defined by:

$$\sigma_1{}' = \sigma_3{}' + \sigma_{ci}\left(m_b \frac{\sigma_3{}'}{\sigma_{ci}} + s\right)^a$$

Generalized Hoek-Brown Criterion

$$m_b = m_i exp\left(\frac{GSI - 100}{28 - 14D}\right)$$

$$s = exp\left(\frac{GSI - 100}{9 - 3D}\right)$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

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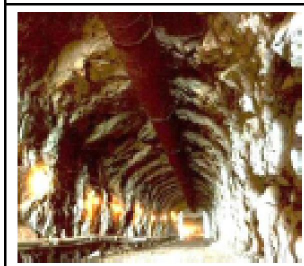
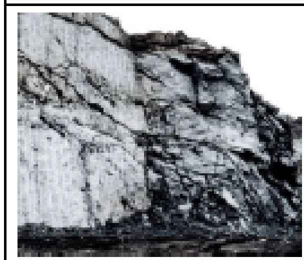
Geological Strength Index (GSI)

D is a factor which depends upon the degree of disturbance due to blast damage and stress relaxation. It varies **from 0** for undisturbed in situ rock masses **to 1** for very disturbed rock masses.

Guidelines for Estimating Disturbance Factor (D)

Hoek, 2007

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	D = 0
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	D = 0 D = 0.5 No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	D = 0.8
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	D = 0.7 Good blasting D = 1.0 Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	D = 1.0 Production blasting D = 0.7 Mechanical excavation

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Geological Strength Index (GSI)

Note that **the factor D applies only to the blast damaged zone** and it should not be applied to the entire rock mass. *For example*, in tunnels the blast damage is generally limited to a 1 to 2 m thick zone around the tunnel and this should be incorporated into numerical models as a different and weaker material than the surrounding rock mass. Applying the blast damage factor D to the entire rock mass is inappropriate and can result in misleading and unnecessarily pessimistic results.

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Generalized Hoek-Brown Criterion

The Generalized Hoek-Brown failure criterion for **jointed rock masses** is defined by:

$$\sigma_1{}' = \sigma_3{}' + \sigma_{ci}\left(m_b\frac{\sigma_3{}'}{\sigma_{ci}} + s\right)^a$$

The **uniaxial compressive strength of the rock mass** is obtained by setting $\sigma_3' = 0$, giving:

$$\sigma_{cm} = \sigma_{ci} \cdot s^a$$

Generalized Hoek-Brown Criterion

$$\sigma_1{}' = \sigma_3{}' + \sigma_{ci}\left(m_b \frac{\sigma_3{}'}{\sigma_{ci}} + s\right)^a$$

and the **tensile strength of the rock mass** is:

$$\sigma_t = -\frac{s\sigma_{ci}}{m_b}$$

It is obtained by setting $\sigma_1' = \sigma_3' = \sigma_t$. This represents a condition of biaxial tension. Hoek (1983) showed that for brittle materials, the uniaxial tensile strength is equal to the biaxial tensile strength.

Geological Strength Index (GSI)

Note that the *switch* at GSI = 25 for the coefficients s and a (Hoek and Brown, 1997) has been eliminated which give smooth continuous transitions for the entire range of GSI values.

THANK YOU

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