# FUNDAMENTALS OF GEOTECHNICAL ENGINEERING

## Lesson 8. Mechanical behaviour of rock masses.

Jesús Mª Hernández
Mª Helena Fernandes
Department of Mechanical Engineering
Faculty of Engineering

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# 8. MECHANICAL BEHAVIOUR OF ROCK MASSES.

## LESSON OVERVIEW

In this lesson, the fundamentals of the mechanical behaviour of rock masses are presented. First, the mechanical behaviour of intact rocks is determined by means of different laboratory tests. It is shown that this behaviour is more complex than expected as a brittle material. Then, a basic analysis of discontinuities is introduced. Finally, the mechanical behaviour of rock masses is explained, based on the behaviour of intact rocks and discontinuities.

## LEARNING OUTCOMES

On completion this lesson, the student will be able to:

- ✓ Know, understand and interpret the laboratory tests to determine the mechanical properties of intact rocks: uniaxial compression test, point load test, Schmidt hammer, Brazilian test and triaxial compression test.
- ✓ Know and understand the failure criteria for intact rocks, Mohr-Coulomb and Hoek-Brown, determine their mathematical expressions from laboratory tests, and apply them to verify whether an intact rock fails.
- ✓ Know and understand the basis of the mechanical behaviour of rock discontinuities.
- ✓ Know and understand the mechanical behaviour of rock masses from the behaviour of their constituents: intact rock and discontinuities.



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# 8. MECHANICAL BEHAVIOUR OF ROCK MASSES.

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## INTRODUCTION

- The mechanical behaviour of rock masses is very complex, but it can be considered that it depends mainly on the following parameters:
    1. Mechanical properties of the intact rock.
    2. Mechanical properties of discontinuities (joints).
    3. Environmental and historical conditions of the rock mass (natural stresses, hydrogeology, etc.).

- Each parameter could be important, depending on the weathering grade of the rock mass.
    - If the weathering grade is very low, the predominant parameter is the first (intact rock).
    - If the weathering grade is medium-low, i.e., there are few families of discontinuities, the predominant parameter is the second.
    - If the weathering grade is high-very high, all the parameters are important.

## MECHANICAL PROPERTIES OF THE INTACT ROCK (I)

### Uniaxial compression test (I) UNE 22950-1:1990 and ASTM D7012

- To determine the *uniaxial compressive strength* of an intact rock ($\sigma_C$ or $q_U$).
- On a cylindrical rock sample a vertical load is applied and steadily increased (0.5-1.0 MPa/s) until the rock specimen fails.
- Five tests are required.
- The uniaxial compressive strength is the load applied at failure divided by the cross-sectional area of the rock sample.

Video: Uniaxial Compression Test.

## MECHANICAL PROPERTIES OF THE INTACT ROCK (II)

### Uniaxial compression test (II)

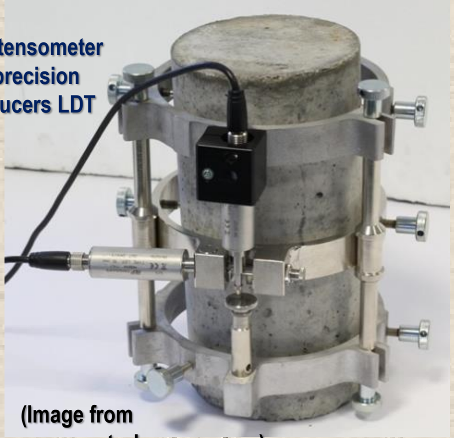
❖ Elastic properties of the intact rock, i.e., elastic modulus (E) and Poisson's ratio (υ) can also be obtained. UNE 22950-3:1990

❖ To determine those parameters, it will be necessary to use strain gauges to measure axial and diametral deformations.


Determination of Elastic modulus using surface-mounted strain gauges

(Image from www.controls-group.com)

Compressometer/extensometer fitted with two high precision displacement transducers LDT



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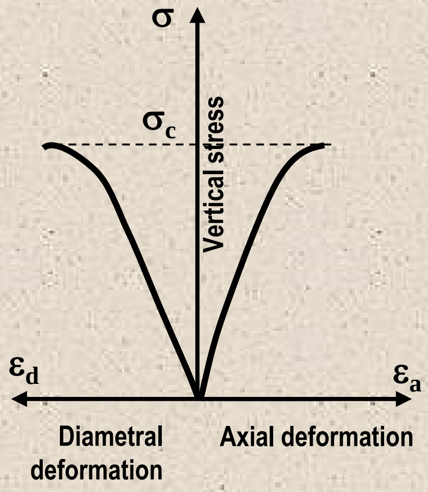
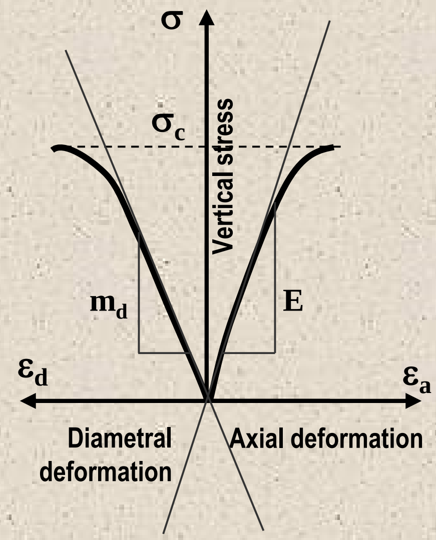


## MECHANICAL PROPERTIES OF THE INTACT ROCK (III)

### Uniaxial compression test (III)

- The elastic modulus (E) is obtained in the figure.
- And the Poisson's ratio (ν) as follows:

$$\upsilon = \frac{-E}{m_d}$$




| Rock Type | E (ksi) [GPa] | Poisson's Ratio |
|---|---|---|
| Igneous | (1450 - 14504) [10 - 100] | 0.10 - 0.40 |
| Granite | (4351 - 10153) [30 - 70] | 0.17 |
| Diorite | (4351 - 14504) [30 - 100] | 0.10 - 0.20 |
| Gabbro | (5802 - 14504) [40 - 100] | 0.20 - 0.35 |
| Rhyolite | (1450 - 7252) [10 - 50] | 0.20 - 0.40 |
| Andesite | (1450 - 10153) [10 - 70] | 0.20 |
| Basalt | (5802 - 11603) [40 - 80] | 0.10 - 0.20 |
| Sedimentary | (725 - 13053) [5 - 90] | 0.10 - 0.30 |
| Conglomerate | (1450 - 13053) [10 - 90] | 0.10 - 0.15 |
| Sandstone | (2176 - 7252) [15 - 50] | 0.14 |
| Shale | (725 - 4351) [5 - 30] | 0.10 |
| Mudstone | (725 - 10153) [5 - 70] | 0.15 |
| Dolomite | (4351 - 10153) [30 - 70] | 0.15 |
| Limestone | (2901 - 10153) [20 - 70] | 0.30 |
| Metamorphic | (725 - 13053) [5 - 90] | 0.15 - 0.30 |
| Gneiss | (4351 - 11603) [30 - 80] | 0.24 |
| Schist | (725 - 8702) [5 - 60] | 0.15 - 0.25 |
| Phyllite | (1450 - 12328) [10 - 85] | 0.26 |
| Slate | (2901 - 13053) [20 - 90] | 0.20 - 0.30 |
| Marble | (4351 - 10153) [30 - 70] | 0.15 - 0.30 |
| Quartzite | (7252 - 13053) [50 - 90] | 0.17 |

Typical properties for various rocks (https://structx.com)

## MECHANICAL PROPERTIES OF THE INTACT ROCK (IV)

### Point load test (I) UNE 22950-5:1996 and ASTM D5731

Video: Point Load Test.

❖ It is used to calculate the rock strength index of a rock specimen. From the rock strength index, the uniaxial compressive strength can be estimated.

❖ A specimen is simply subjected to an increasingly concentrated load until failure occurs. The load is applied in a concentrated manner using two coaxial, conical platens.


(Image from www.controls-group.com)

❖ There are no specifications for the shape of the specimen, as diametral, axial, block, and irregular shapes may be used (at least 10).

❖ The uncorrected point load strength index is determined as

$$\boxed{I_S = \frac{P}{D_e^2}}$$

where P is the maximum applied load, and $D_e$ is the equivalent core diameter, which depends on the type of test (axial, diametral, …) and on the shape of the sample.

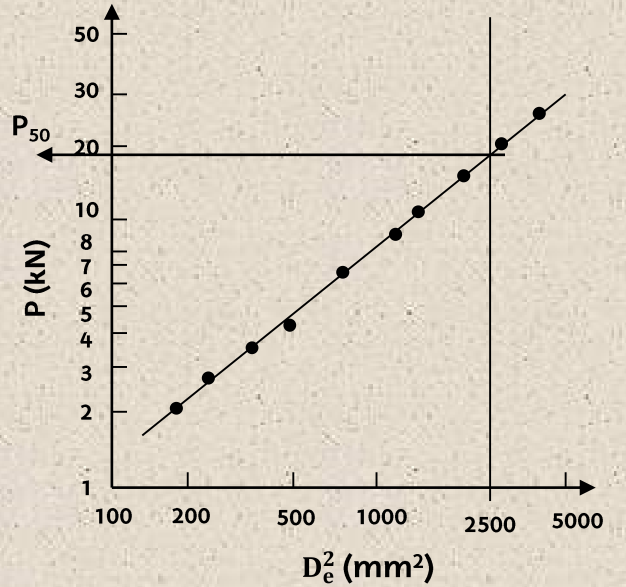
## MECHANICAL PROPERTIES OF THE INTACT ROCK (V)

### Point load test (II)

- From $I_S$, the ***point load strength index***, $I_{S(50)}$, is obtained as

$$I_{S(50)} = \frac{P_{50}}{50^2}$$

where $P_{50}$ can be determined from the figure by interpolation/extrapolation, using the results from the tests.




- Finally, the uniaxial compressive strength can be estimated as:

$$\boxed{\sigma_C \approx 23 \cdot I_{S(50)}}$$

## MECHANICAL PROPERTIES OF THE INTACT ROCK (VI)

### Schmidt hammer (sclerometer)

- To estimate the uniaxial compressive strength of a rock.
- The main parts of the hammer are a mass-spring system and a plunger.
- The hammer measures the rebound of a spring-loaded mass impacting against the surface of a sample. The test hammer hits the rock at a defined energy. Its rebound is dependent on the hardness of the rock and is measured by the test equipment ("rebound number").
- Using the chart provided by the manufacturer, the rebound number and the unit weight of the rock sample, the uniaxial compressive strength of the rock is estimated.

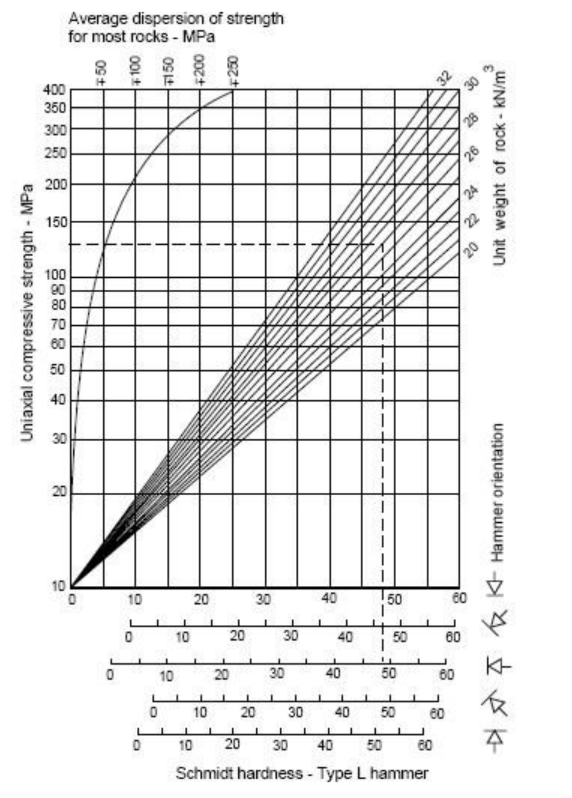



Chart provided by Controls Group
(rock classification hammer)
www.controls-group.com

## MECHANICAL PROPERTIES OF THE INTACT ROCK (VII)

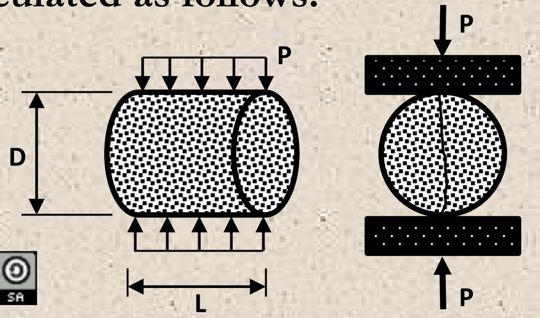
### Brazilian test

UNE 22950-2:1990 and ASTM D7012

Video: Brazilian Test.

- The Brazilian test is a geotechnical laboratory test for indirect measurement of *tensile strength of rocks*.
- A disc shape specimen of the rock is loaded by two opposing normal strip loads at the disc periphery. The specimen diameter shall preferably be not less than 54 mm, or at least 10 times the average grain size. The thickness/diameter ratio should be 0.5 to 0.6. The load is continuously increased at a constant rate until failure of the sample occurs within few minutes. The loading rate depends on the material and may vary from 10 to 50 kN/min. At the failure, the tensile strength of the rock is calculated as follows.

$$\sigma_t = \frac{2P}{\pi \cdot D \cdot L}$$




- 5 tests are required.

## MECHANICAL PROPERTIES OF THE INTACT ROCK (VIII)

### Triaxial compression test

UNE 22950-4:1992 and ASTM D7012

It is quite similar to the triaxial compression test in soils. To apply the confining pressure, oil is used instead of water.


(Image from www.controls-group.com)

- 3 to 5 tests are required.
- In each test the vertical stress at failure ($\sigma_1$) and the confining pressure ($\sigma_3$ or p) are known, i.e., the Mohr's circle at failure.
- From these results, the strength parameters necessary to define the mechanical behaviour of the intact rock can be determined.

## FAILURE CRITERIA FOR INTACT ROCKS (I)

### Mohr-Coulomb criterion (I)

It is a linear criterion defined as

$$\boxed{\tau_f = c + \sigma \cdot tg\phi}$$

c, cohesion, is the force between minerals that constitute the rock.

$\phi$, is the friction angle between two planes of the rock.

These parameters change depending on the directions of the applied force and the anisotropy planes in the rock, and are determined by means of the triaxial compression test.

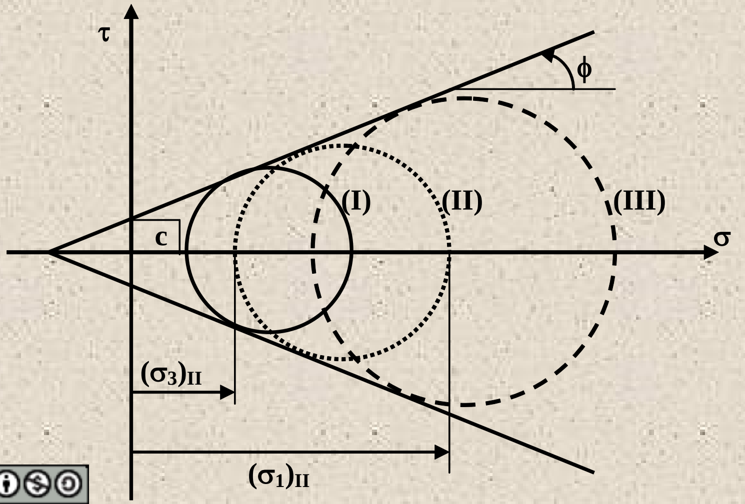
## FAILURE CRITERIA FOR INTACT ROCKS (II)

### Mohr-Coulomb criterion (II)

❖ In each triaxial test the vertical stress at failure ($\sigma_1$) and the confining pressure ($\sigma_3$ or p) are determined, i.e., the Mohr's circle at failure. 3 to 5 tests are required.

❖ Mohr's circles are plotted and an envelope (a straight line) is obtained. Then, cohesion (c) and friction angle ($\phi$) are determined.

| Rock | c (MPa) | $\phi$ |
|---|---|---|
| Basalt | 20-60 | 48-55 |
| Gneiss | 15-40 | 30-40 |
| Granite | 15-50 | 45-58 |
| Limestone | 5-40 | 35-50 |
| Quartzite | 25-70 | 40-55 |
| Sandstone | 8-35 | 30-50 |
| Tufa | 0.7 | - |



## FAILURE CRITERIA FOR INTACT ROCKS (III)

### Hoek & Brown criterion (I)

It is an empirical non-linear criterion defined as follows:

$$\sigma_1 = \sigma_3 + \sqrt{m_i \cdot \sigma_{ci} \cdot \sigma_3 + \sigma_{ci}^2}$$

$\sigma_1$ and $\sigma_3$ are the principal stresses at failure.

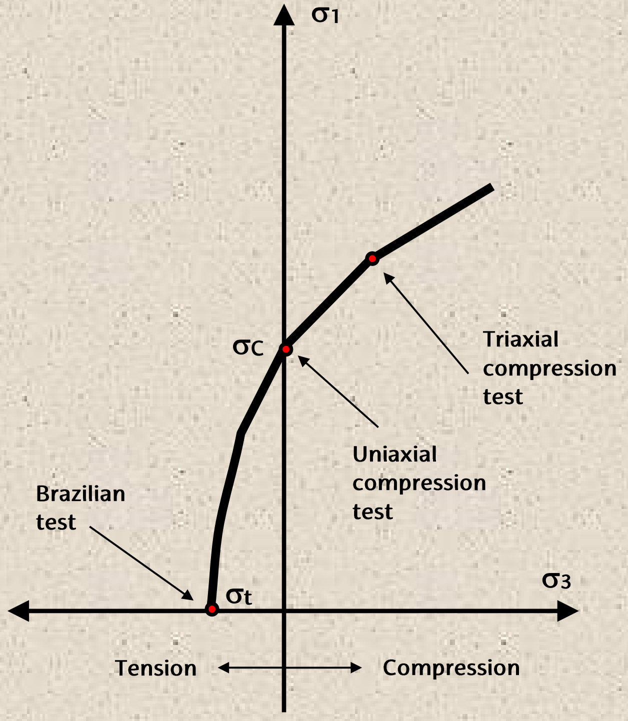
$\sigma_{ci}$ is the uniaxial compressive strength of the intact rock ($\sigma_c$ or $q_U$).

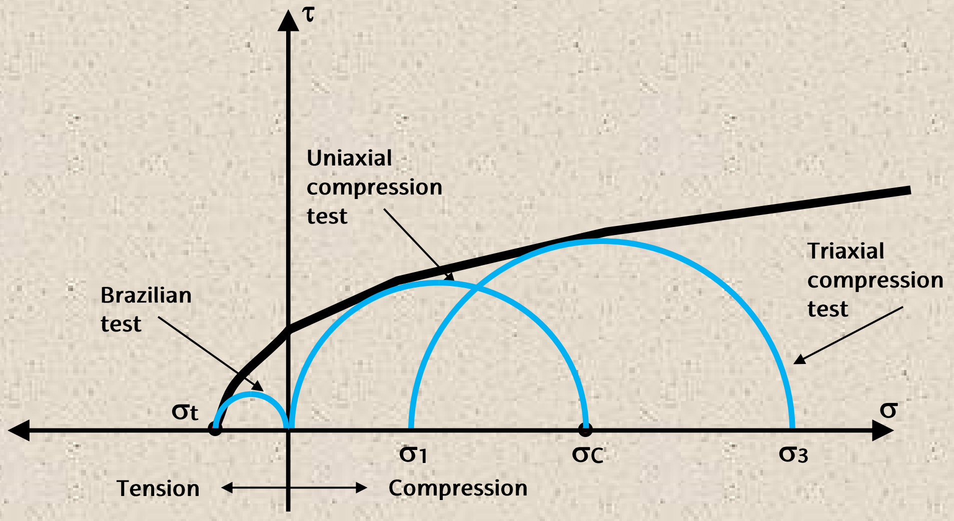
$m_i$ is a constant depending on the properties of the intact rock.

This criterion is more adequate than the Mohr-Coulomb criterion. It also needs the results from the triaxial compression test. In each test the vertical stress at failure ($\sigma_1$) and the confining pressure ($\sigma_3$) are obtained. By combining these stresses, $m_i$ and $\sigma_{ci}$ can be determined.

# FAILURE CRITERIA FOR INTACT ROCKS (IV)

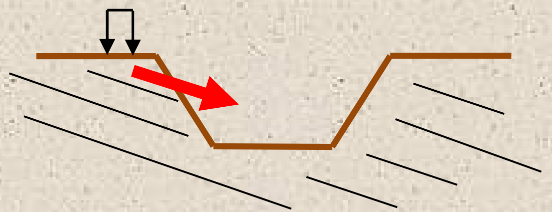
## Hoek & Brown criterion (II)



## DISCONTINUITIES (I)

❖ They are weak surfaces in the rock masses. Failure (sliding) can occur along these surfaces.



(Own work)

❖ Discontinuities cause anisotropy in rock masses, and are characterized by:

➢ Orientation (favourable/unfavourable).

➢ Opening, roughness/smoothness, infilling, separation, length (persistence), and groundwater conditions (Tables D.10 to D.17 from TBC.Foundations).

## DISCONTINUITIES (II)

### Mechanical behaviour

- It is defined by the shear strength of the discontinuity, i.e., by cohesion (c) and friction angle (ϕ), according to the Mohr-Coulomb criterion.
- It mainly depends on its roughness, imbrication (fitting), and infilling.
- It is determined by means of laboratory or in situ tests.

Shear box apparatus
(Image from www.controls-group.com)

## FAILURE CRITERIA FOR ROCK MASSES (I)

Four possibilities can be distinguished, depending on the weathering grade of the rock mass:

1. Rock mass without discontinuities ≅ Intact rock.

   Failure appears in the intact rock. In this case, any of both criteria (Mohr-Coulomb and Hoek & Brown) can be used.

2. Rock mass with 1 or 2 families of discontinuities.

   Failure can appear whether along the discontinuities or in the intact rock. Then, it is necessary to study both possibilities.

   - In order to study the first possibility, cohesion (c) and friction angle (ϕ) of discontinuities should be known. Then, Mohr-Coulomb criterion will be used.
   - To analyse the second possibility, any of both criteria (Mohr-Coulomb and Hoek & Brown) will be used. So, triaxial compression tests are necessary.

## FAILURE CRITERIA FOR ROCK MASSES (II)

Four possibilities can be distinguished, depending on the weathering grade of the rock mass:

3. Rock mass with three (3) families of discontinuities.

   Failure can only appear **along the discontinuities**. Then, the Mohr-Coulomb criterion for discontinuities is used.

4. Rock mass with four (4) or more families of discontinuities.

   Due to the high number of discontinuities, it can be assumed **an isotropic behaviour of the rock mass**. Then the Hoek & Brown criterion for rock masses is used (next slide).

## FAILURE CRITERIA FOR ROCK MASSES (III)

### Hoek & Brown criterion

It is an extension if the same criterion for intact rocks.

$$\boxed{\sigma_1 = \sigma_3 + \sigma_{ci} \cdot \left( m \cdot \frac{\sigma_3}{\sigma_{ci}} + s \right)^a} \Leftarrow \boxed{\sigma_1 = \sigma_3 + \sqrt{m_i \cdot \sigma_{ci} \cdot \sigma_3 + \sigma_{ci}^2}}$$

m is the value of the Hoek & Brown constant $m_i$ for the rock mass.

s and a are constants which depend on the rock mass characteristics.

$$m = m_i \cdot e^{\left(\frac{GSI-100}{28}\right)}$$

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

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

GSI is a geomechanical classification

D is a factor which depends upon the degree of disturbance due to blast damage and stress relaxation.

## DEFORMABILITY OF ROCK MASSES

- Its real calculation is very complex.
- It depends on the deformation of the intact rock and discontinuities, and it is always larger than the first one.
- According to the TBC.Foundations, in order to determine it, the theory of elasticity must be used. Then, the main parameter is the elastic modulus of the rock mass ($E_{rockmass}$).
- As an indication, if the RMR of the rock mass is less than 50, the following expression can be used to determine $E_{rockmass}$

$$\boxed{E_{rockmass} = \alpha \cdot 10^{\frac{RMR-10}{40}} \ (GPa)}$$

where

$\alpha = 0.1$, when $q_U = 1$ MPa
$\alpha = 0.2$, when $q_U = 4$ MPa
$\alpha = 0.3$, when $q_U = 10$ MPa
$\alpha = 0.7$, when $q_U = 50$ MPa
$\alpha = 1.0$, when $q_U > 100$ MPa

- For good quality rock masses, having RMR > 55, Bienawski proposed the following expression:

$E_{rockmass} = 2 \cdot RMR - 100$ (GPa).

## BEARING CAPACITY OF ROCK MASSES

### TBC.Foundations

➔ If $q_U$ < 2.5 MPa or RQD < 25% or the weathering grade is V or VI, the rock mass will be considered as a soil.

➔ For the rest of the cases, $q_{allow}$ can be determined as: $\boxed{q_{allow} = K_{sp} \cdot q_U}$

following these conditions:

- ➔ ground surface is almost horizontal.
- ➔ if there is an applied horizontal load, its magnitude should be less than 10 % of vertical load.
- ➔ if it is a sedimentary rock, the strata must be horizontal.

$$K_{sp} = \frac{3 + \frac{s}{B}}{10 \cdot \sqrt{1 + 300 \cdot \frac{a}{s}}}$$

s, separation between discontinuities, s > 30 cm

B, width of the foundation; 0.05 < s/B < 2

a, opening of discontinuities; a < 5 mm where the joint is empty, a < 25 mm where the joint is filled; 0 < a/s < 0.02