Mecánica de Rocas en obras de Ingeniería (CI52-T)

Profesores:
Ricardo Moffat
Sergio Sepúlveda

Contenidos

- 3.0. Resistencia de la roca intacta
  - 3.1. Concepto de roca intacta y propiedades características.
  - 3.2. Tensores de esfuerzo y deformación.
  - 3.3. Criterios de falla

Table 1 : Typical problems, critical parameters, methods of analysis and acceptability criteria for slopes.

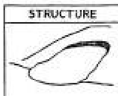
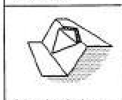
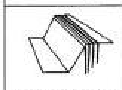
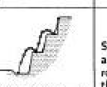
STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Landslides.	Complex failure along a circular or near circular failure surface involving sliding on faults and other structural features as well as failure of intact materials.	• Presence of regional faults. • Shear strength of materials along failure surface. • Groundwater distribution in slope, particularly in response to rainfall or to submergence of slope toe. • Potential earthquake loading.	Limit equilibrium methods which allow for non-circular failure surfaces can be used to estimate changes in factor of safety as a result of drainage or slope profile changes. Numerical methods such as finite element or discrete element analysis can be used to investigate failure mechanisms and history of slope displacement.	Absolute value of factor of safety has little meaning but rate of change of factor of safety can be used to judge effectiveness of remedial measures. Long term monitoring of surface and subsurface displacements in slope is the only practical means of evaluating slope behaviour and effectiveness of remedial action.
 Soil or heavily jointed rock slopes.	Circular failure along a spoon-shaped surface through soil or heavily jointed rock masses.	• Height and angle of slope face. • Shear strength of materials along failure surface. • Groundwater distribution in slope. • Potential surcharge or earthquake loading.	Two-dimensional limit equilibrium methods which include automatic searching for the critical failure surface are used for parametric studies of factor of safety. Probability analyses, three-dimensional limit equilibrium analyses or numerical stress analyses are occasionally used to investigate unusual slope problems.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Where displacements are critical, numerical analyses of slope deformation may be required and higher factors of safety will generally apply in these cases.
 Jointed rock slopes.	Planar or wedge sliding on one structural feature or along the line of intersection of two structural features.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading. • Sequence of excavation and support installation.	Limit equilibrium analyses which determine three-dimensional sliding modes are used for parametric studies on factor of safety. Failure probability analyses, based upon distribution of structural orientations and shear strengths, are useful for some applications.	Factor of safety > 1.3 for "temporary" slopes with minimal risk of damage. Factor of safety > 1.5 for "permanent" slopes with significant risk of damage. Probability of failure of 10 to 15% may be acceptable for open pit mine slopes where cost of clean up is less than cost of stabilization.
 Vertically jointed rock slopes .	Toppling of columns separated from the rock mass by steeply dipping structural features which are parallel or nearly parallel to the slope face.	• Slope height, angle and orientation. • Dip and strike of structural features. • Groundwater distribution in slope. • Potential earthquake loading.	Crude limit equilibrium analyses of simplified block models are useful for estimating potential for toppling and sliding. Discrete element models of simplified slope geometry can be used for exploring toppling failure mechanisms.	No generally acceptable criterion for toppling failure is available although potential for toppling is usually obvious. Monitoring of slope displacements is the only practical means of determining slope behaviour and effectiveness of remedial measures.
 Loose boulders on rock slopes.	Sliding, rolling, falling and bouncing of loose rocks and boulders on the slope.	• Geometry of slope. • Presence of loose boulders. • Coefficients of restitution of materials forming slope. • Presence of structures to arrest falling and bouncing rocks.	Calculation of trajectories of falling or bouncing rocks based upon velocity changes at each impact is generally adequate. Monte Carlo analyses of many trajectories based upon variation of slope geometry and surface properties give useful information on distribution of fallen rocks.	Location of fallen rock or distribution of a large number of fallen rocks will give an indication of the magnitude of the potential rockfall problem and of the effectiveness of remedial measures such as draped mesh, catch fences and ditches at the toe of the slope.

Table 2 : Typical problems, critical parameters, methods of analysis and acceptability criteria for dams and foundations.

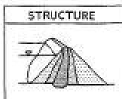
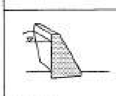
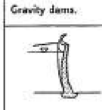
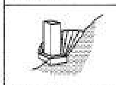
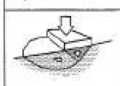
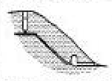
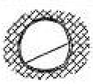
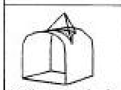
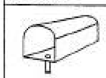
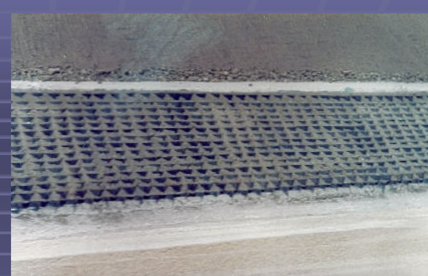
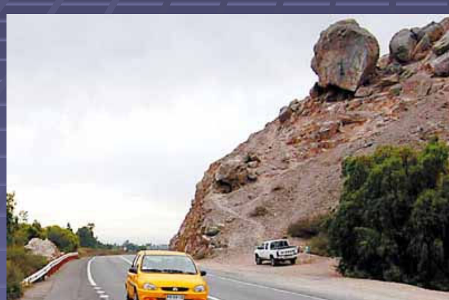
STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Zoned fill dams.	Circular or near-circular failure of dam, particularly during rapid drawdown. Foundation failure on weak seams. Piping and erosion of core.	• Presence of weak or permeable zones in foundation. • Shear strength, durability, gradation and placement of dam construction materials, particularly filters. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Seepage analyses are required to determine water pressure and velocity distribution through dam and abutments. Limit equilibrium methods should be used for parametric studies of stability. Numerical methods can be used to investigate dynamic response of dam during earthquakes.	Safety factor >1.5 for full pool with steady state seepage; >1.3 for end of construction with no reservoir loading and undissipated foundation porewater pressures; >1.2 for probable maximum flood with steady state seepage and >1.0 for full pool with steady state seepage and maximum credible horizontal psuedo-static seismic loading.
 Gravity dams.	Shear failure of interface between concrete and rock or of foundation rock. Tension crack formation at heel of dam. Leakage through foundation and abutments.	• Presence of weak or permeable zones in rock mass. • Shear strength of interface between concrete and rock. • Shear strength of rock mass. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Parametric studies using limit equilibrium methods should be used to investigate sliding on the interface between concrete and rock and sliding on weak seams in the foundation. A large number of trial failure surfaces are required unless a non-circular failure analysis with automatic detection of critical failure surfaces is available.	Safety factor against foundation failure should exceed 1.5 for normal full pool operating conditions provided that conservative shear strength values are used ($c'=0$). Safety factor > 1.3 for probable maximum flood (PMF). Safety factor > 1 for extreme loading - maximum credible earthquake and PMF.
 Arch dams.	Shear failure in foundation or abutments. Cracking of arch due to differential settlements of foundation. Leakage through foundations or abutments.	• Presence of weak, deformable or permeable zones in rock mass. • Orientation, inclination and shear strength of structural features. • Effectiveness of grout curtain and drainage system. • Stability of reservoir slopes.	Limit equilibrium methods are used for parametric studies of three-dimensional sliding modes in the foundation and abutments, including the influence of water pressures and reinforcement. Three-dimensional numerical analyses are required to determine stresses and displacements in the concrete arch.	Safety factor against foundation failure >1.5 for normal full pool operating conditions and >1.3 for probable maximum flood conditions provided that conservative shear strength values are used ($c'=0$). Stresses and deformations in concrete arch should be within allowable working levels defined in concrete specifications.
 Foundations on rock slopes.	Slope failure resulting from excessive foundation loading. Differential settlement due to anisotropic deformation properties of foundation rocks.	• Orientation, inclination and shear strength of structural features in rock mass forming foundation. • Presence of inclined layers with significantly different deformation properties. • Groundwater distribution in slope.	Limit equilibrium analyses of potential planar or wedge failures in the foundation or in adjacent slopes are used for parametric studies of factor of safety. Numerical analyses can be used to determine foundation deformation, particularly for anisotropic rock masses.	Factor of safety against sliding of any potential foundation wedges or blocks should exceed 1.5 for normal operating conditions. Differential settlement should be within limits specified by structural engineers.
 Foundations on soft rock or soil.	Bearing capacity failure resulting from shear failure of soils or weak rocks underlying foundation slab.	• Shear strength of soil or jointed rock materials. • Groundwater distribution in soil or rock foundation. • Foundation loading conditions and potential for earthquake loading.	Limit equilibrium analyses using inclined slices and non-circular failure surfaces are used for parametric studies of factor of safety. Numerical analyses may be required to determine deformations, particularly for anisotropic foundation materials.	Bearing capacity failure should not be permitted for normal loading conditions. Differential settlement should be within limits specified by structural engineers.

Table 3 : Typical problems, critical parameters, methods of analysis and acceptability criteria for underground civil engineering excavations.

STRUCTURE	TYPICAL PROBLEMS	CRITICAL PARAMETERS	ANALYSIS METHODS	ACCEPTABILITY CRITERIA
 Pressure tunnels in hydro-power projects.	Excessive leakage from unlined or concrete lined tunnels. Rupture or buckling of steel lining due to rock deformation or external pressure.	• Ratio of maximum hydraulic pressure in tunnel to minimum principal stress in the surrounding rock. • Length of steel lining and effectiveness of grouting. • Groundwater levels in the rock mass.	Determination of minimum cover depths along pressure tunnel route from accurate topographic maps. Stress analyses of sections along and across tunnel axis. Comparison between minimum principal stresses and maximum dynamic hydraulic pressure to determine steel lining lengths.	Steel lining is required where the minimum principal stress in the rock is less than 1.3 times the maximum static head for typical hydroelectric operations or 1.15 for operations with very low dynamic pressures. Hydraulic pressure testing in boreholes at the calculated ends of the steel lining is essential to check the design assumptions.
 Soft rock tunnels.	Rock failure where strength is exceeded by induced stresses. Swelling, squeezing or excessive closure if support is inadequate.	• Strength of rock mass and of individual structural features. • Swelling potential, particularly of sedimentary rocks. • Excavation method and sequence. • Capacity and installation sequence of support systems.	Stress analyses using numerical methods to determine extent of failure zones and probable displacements in the rock mass. Rock-support interaction analyses using closed-form or numerical methods to determine capacity and installation sequence for support and to estimate displacements in the rock mass.	Capacity of installed support should be sufficient to stabilize the rock mass and to limit closure to an acceptable level. Tunnelling machines and internal structures must be designed for closure of the tunnel as a result of swelling or time-dependent deformation. Monitoring of deformations is an important aspect of construction control.
 Shallow tunnels in jointed rock.	Gravity driven falling or sliding wedges or blocks defined by intersecting structural features. Unravelling of inadequately supported surface material.	• Orientation, inclination and shear strength of structural features in the rock mass. • Shape and orientation of excavation. • Quality of drilling and blasting during excavation. • Capacity and installation sequence of support systems.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass surrounding the tunnel. Limit equilibrium analyses of critical wedges are used for parametric studies on the mode of failure, factor of safety and support requirements.	Factor of safety, including the effects of reinforcement, should exceed 1.5 for sliding and 2.0 for falling wedges and blocks. Support installation sequence is critical and wedges or blocks should be identified and supported before they are fully exposed by excavation. Minimum deformation. Displacement monitoring is of little value.
 Large caverns in jointed rock.	Gravity driven falling or sliding wedges or tensile and shear failure of rock mass, depending upon spacing of structural features and magnitude of in situ stresses.	• Shape and orientation of cavern in relation to orientation, inclination and shear strength of structural features in the rock mass. • In situ stresses in the rock mass. • Excavation and support sequence and quality of drilling and blasting.	Spherical projection techniques or analytical methods are used for the determination and visualization of all potential wedges in the rock mass. Stresses and displacements induced by each stage of cavern excavation are determined by numerical analyses and are used to estimate support requirements for the cavern roof and walls.	An acceptable design is achieved when numerical models indicate that the extent of failure has been controlled by installed support, that the support is not overstressed and that the displacements in the rock mass stabilize. Monitoring of displacements is essential to confirm design predictions.
 Underground nuclear waste disposal.	Stress and/or thermally induced spalling of the rock surrounding the excavations resulting in increased permeability and higher probability of radioactive leakage.	• Orientation, inclination, permeability and shear strength of structural features in the rock mass. • In situ and thermal stresses in the rock surrounding the excavations. • Groundwater distribution in the rock mass.	Numerical analyses are used to calculate stresses and displacements induced by excavation and by thermal loading from waste canisters. Groundwater flow patterns and velocities, particularly through blast damaged zones, fissures in the rock and shaft seals are calculated using numerical methods.	An acceptable design requires extremely low rates of groundwater movement through the waste canister containment area in order to limit transport of radioactive material. Shafts, tunnels and canister holes must remain stable for approximately 50 years to permit retrieval of waste if necessary.





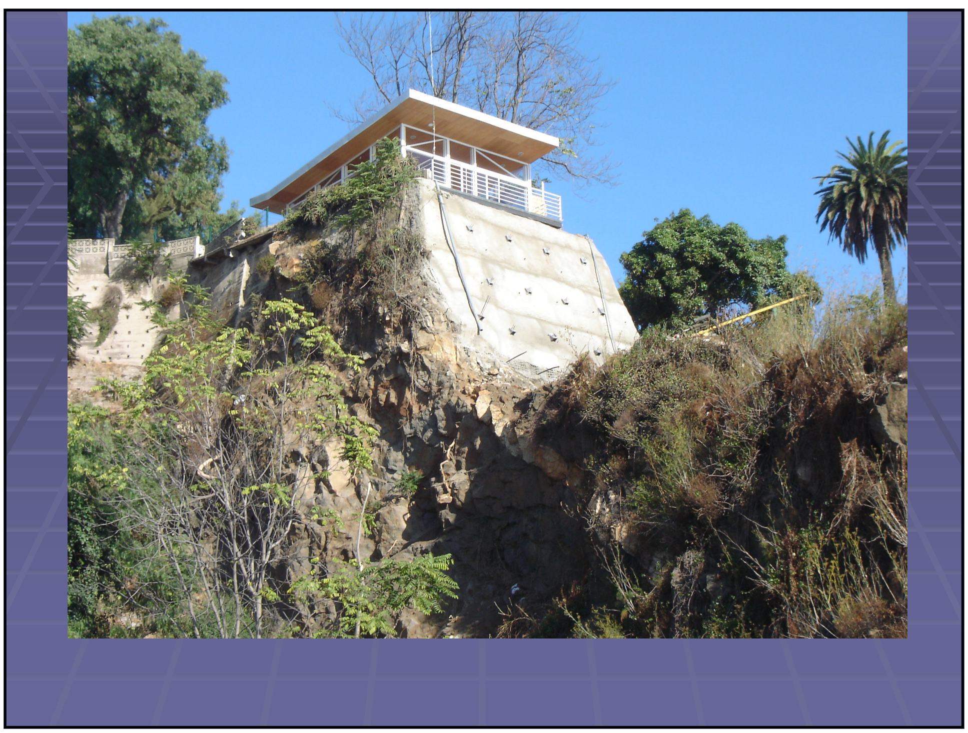
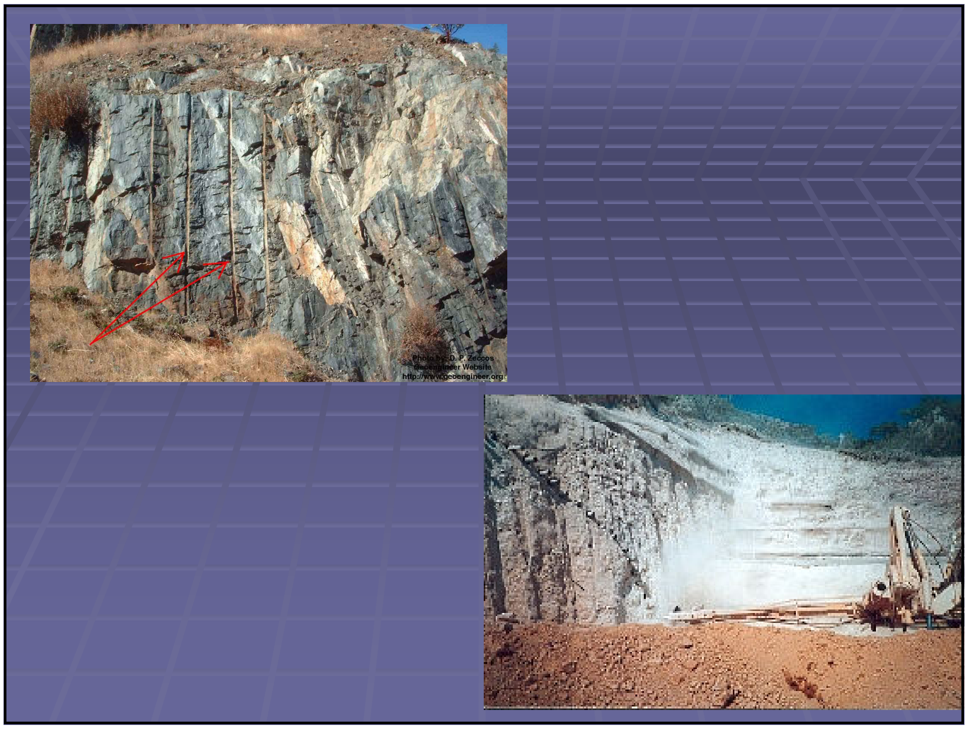
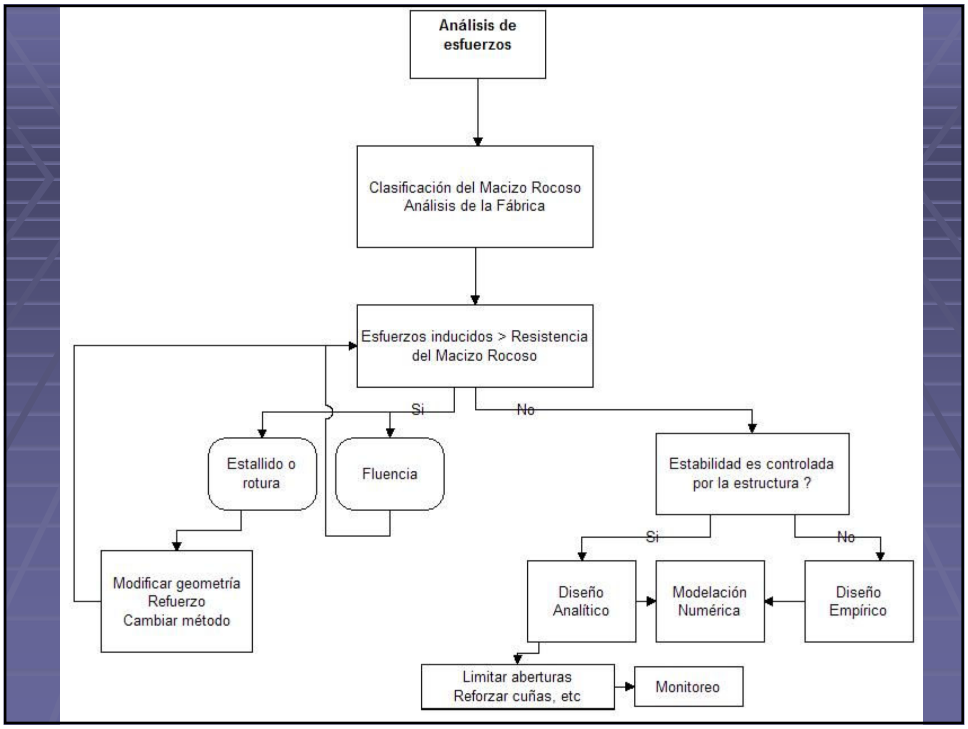



Photo by: EDAFOMICHANIKI Ltd
Available by the Geoengineer Website
http://www.geoengineer.org





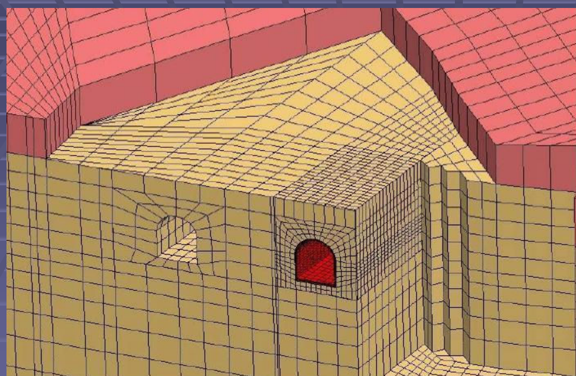
Elementos Importantes

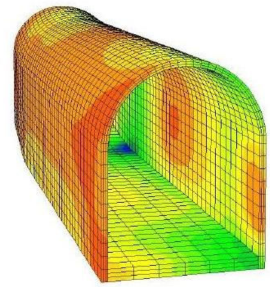
- Composición de la roca (litología, mineralogía, composición química)
- Estructuras geológicas
- Resistencia de la roca y las estructuras
- Resistencia del macizo rocoso
- Deformabilidad
- Densidad
- Porosidad y Permeabilidad
- Estado de esfuerzos

Tensiones en la roca

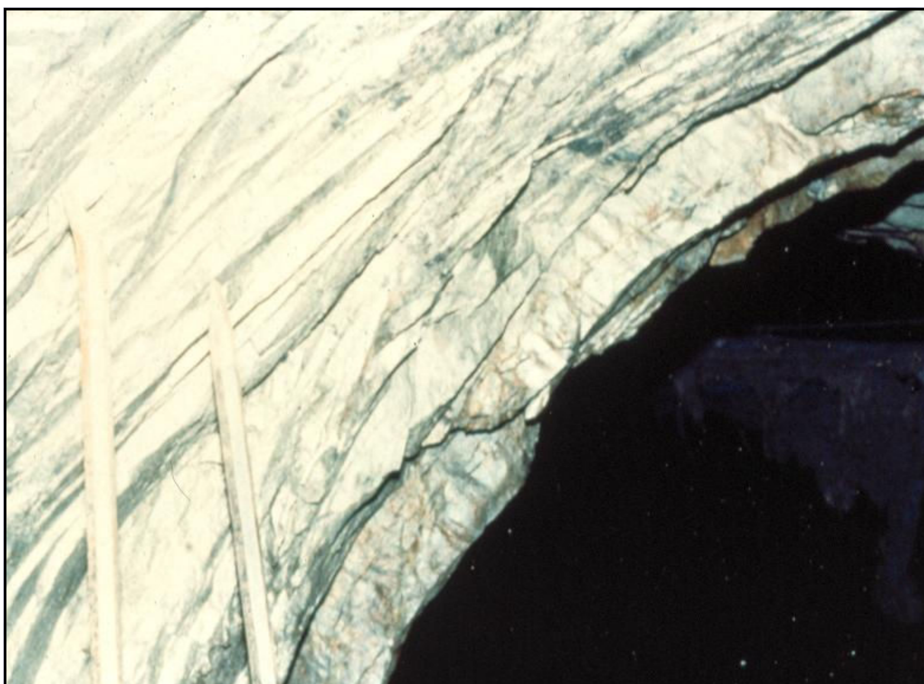
- Pre-existentes
  - Peso propio del macizo rocoso
  - Origen tectónico
- Cambio de esfuerzos debido a una obra de ingeniería

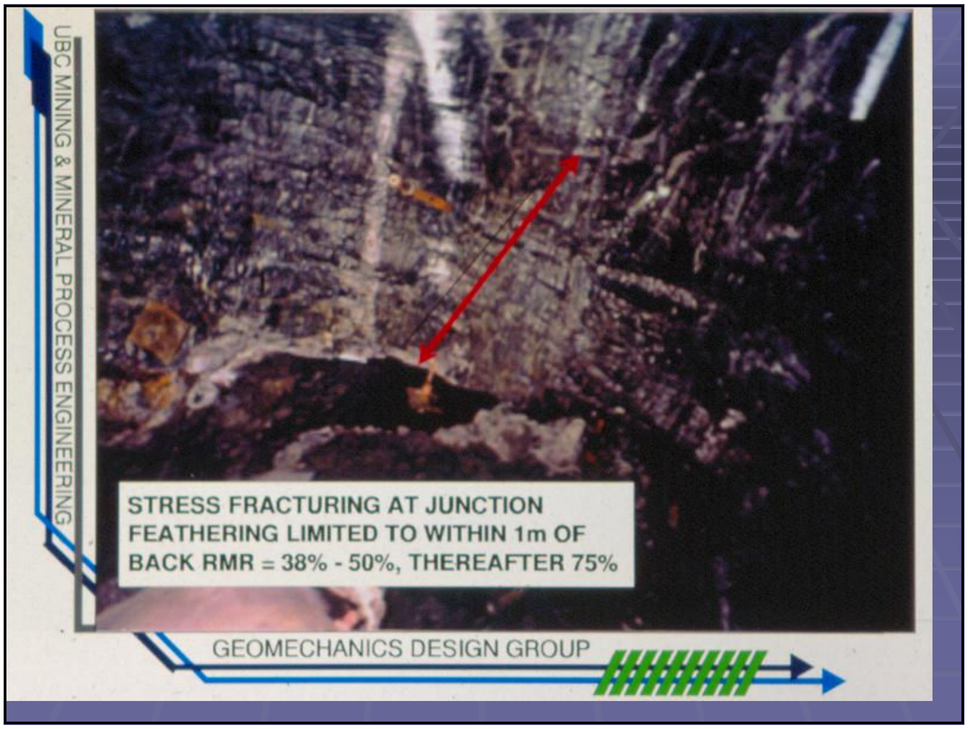
El estado tensional en la roca se puede caracterizar por un tensor de esfuerzos

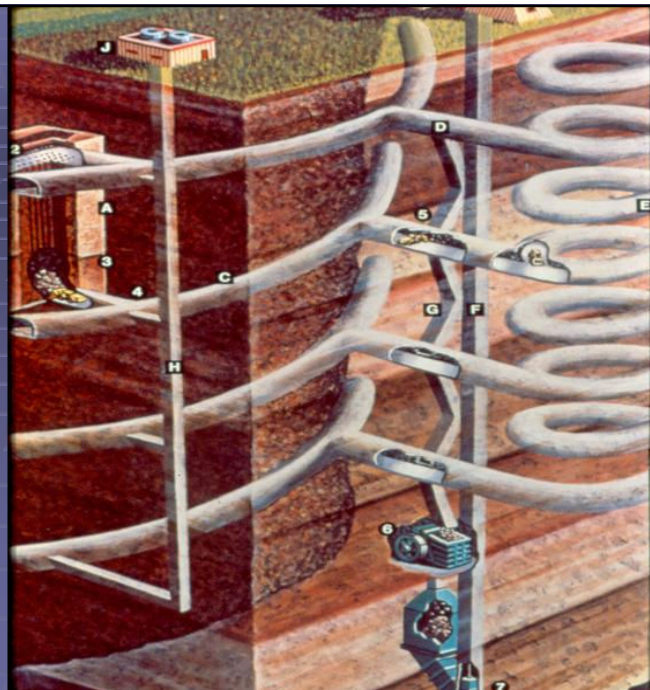




Calculated stresses within the concrete lining as a result of the adjacent excavation.

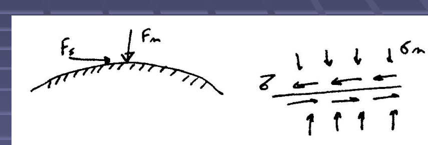


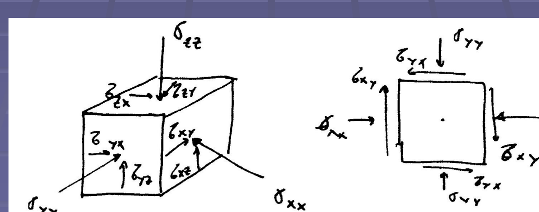

UBC MINING & MINERAL PROCESS ENGINEERING
STRESS FRACTURING AT JUNCTION
FEATHERING LIMITED TO WITHIN 1m OF
BACK RMR = 38% - 50%, THEREAFTER 75%
GEOMECHANICS DESIGN GROUP



Tensor de esfuerzos

- En un plano cualquiera pueden existir fuerzas normales y de corte.



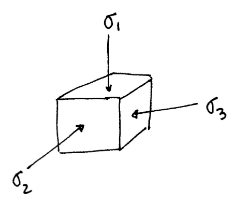


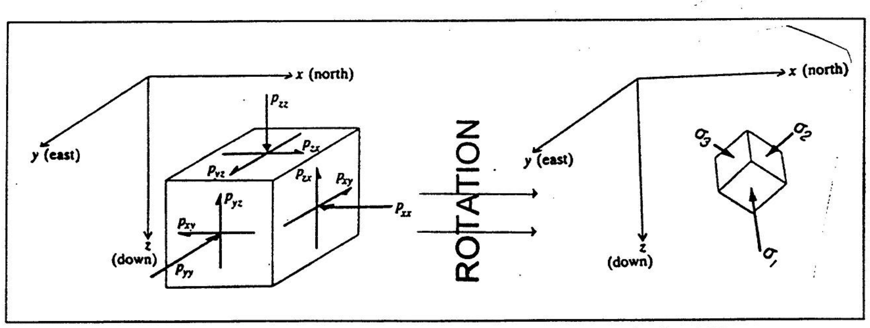
$$\begin{bmatrix} \sigma_{xx} & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_{yy} & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_{zz} \end{bmatrix}$$

El estado tensional tiene 6 componentes independientes (tensor de esfuerzos)

Existe una orientación en la cual no existe corte. Planos principales o esfuerzos principales.

$$\begin{bmatrix} \sigma_1 & 0 & 0 \\ 0 & \sigma_2 & 0 \\ 0 & 0 & \sigma_3 \end{bmatrix}$$



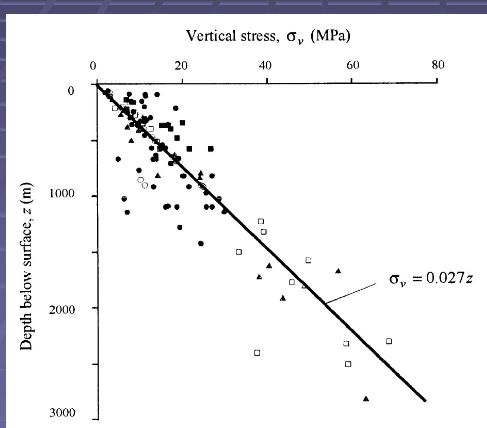


Transformation of Normal and Shear Stresses to Principal Stresses

Esfuerzos in-situ

- Métodos de determinación de esfuerzos in-situ
  - Mediciones
  - Métodos indirectos
    - Teoría de la elasticidad

Predicción de esfuerzos a través de la teoría de la elasticidad

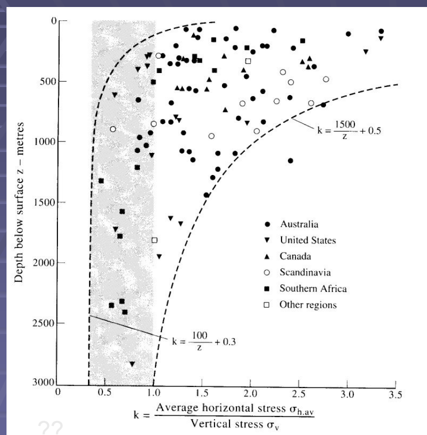


Tensión vertical

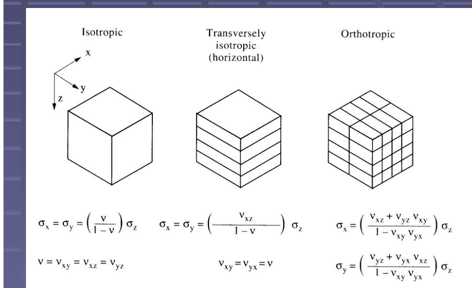
$$\sigma_v = \gamma \cdot z$$

Hoek and Brown (1978)

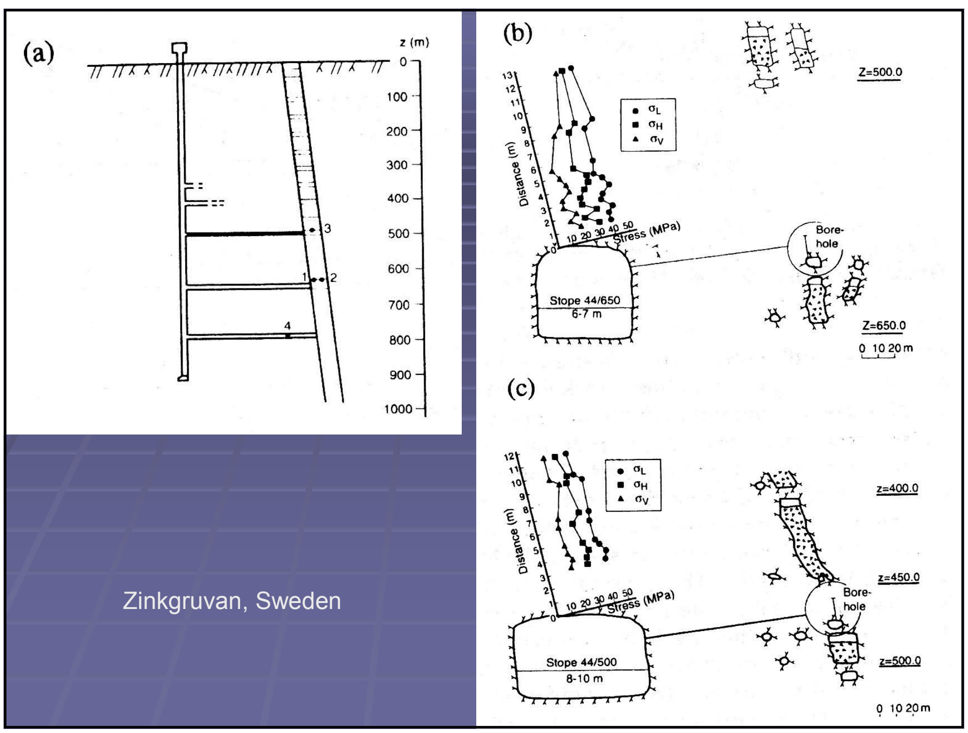
Esfuerzos horizontales



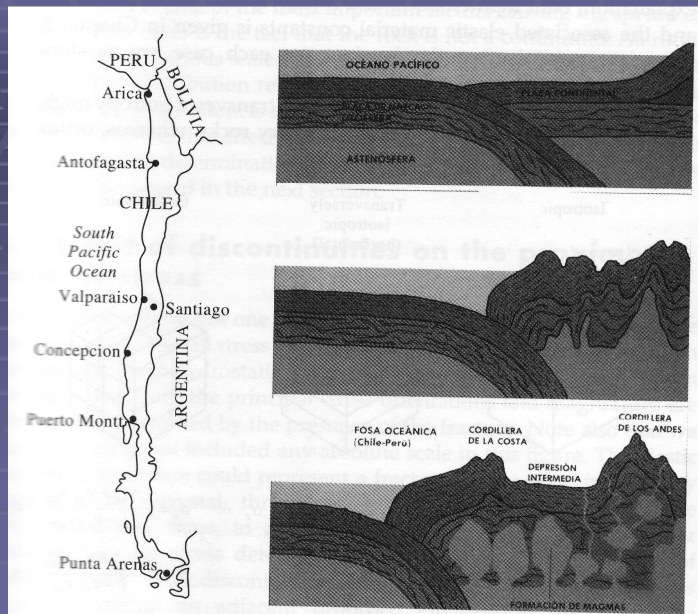
Hoek and Brown (1980)

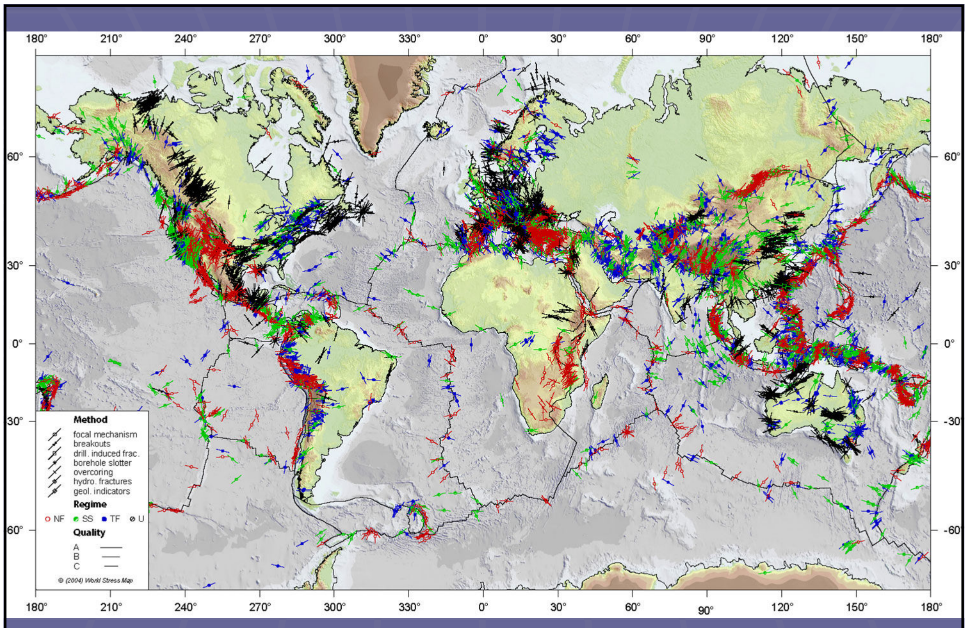


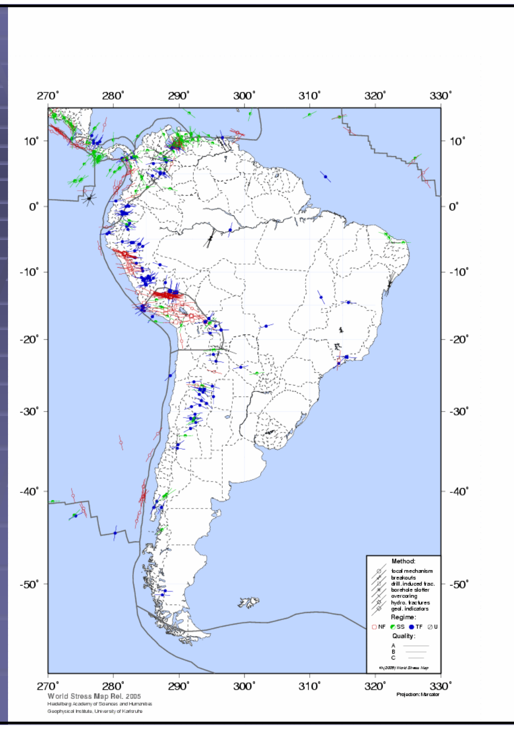
Hudson y Harrison (1997)



Zinkgruvan, Sweden


PERU
BOLIVIA
Arica
Antofagasta
CHILE
South Pacific Ocean
Valparaiso
Santiago
Concepcion
ARGENTINA
Puerto Montt
Punta Arenas
OCÉANO PACÍFICO
PLACA CONTINENTAL
ASTENÓSFERA
FOSA OCEÁNICA (Chile-Perú)
CORDILLERA DE LA COSTA
CORDILLERA DE LOS ANDES
DEPRESIÓN INTERMEDIA
FORMACIÓN DE MAGMAS


180° 210° 240° 270° 300° 330° 0° 30° 60° 90° 120° 150° 180°
60° 30° 0° -30° -60°
Method
focal mechanism
breakouts
drill. induced frac.
borehole slotter
overcoring
hydro. fractures
geol. indicators
Regime
NF SS TF U
Quality
A
B
C
© (2004) World Stress Map

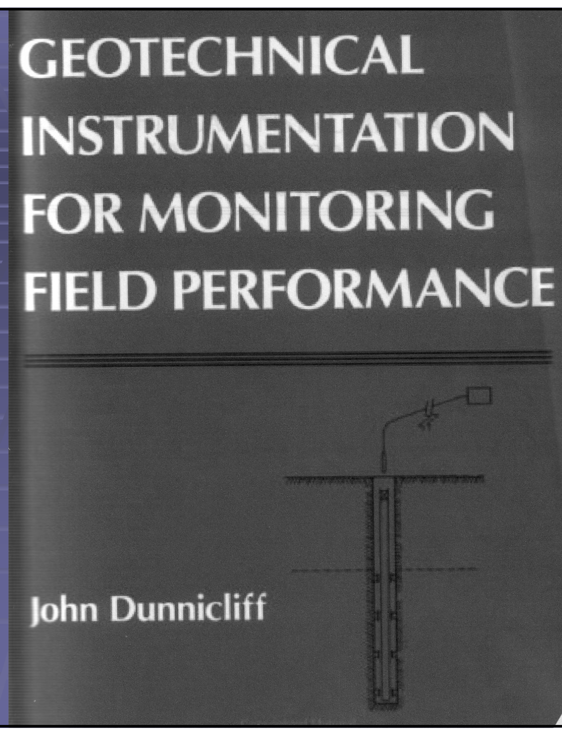


Métodos para la determinación del estado tensional

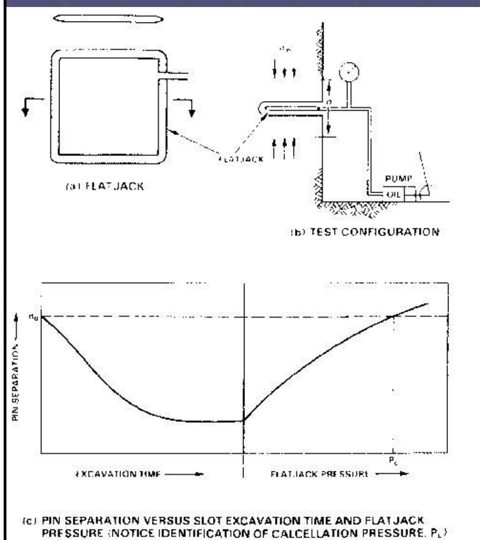
- Flatjack
- Método de la fractura hidráulica
- Métodos de deformación

Métodos para predecir esfuerzos in situ

- Son métodos indirectos
- Se basan en algún tipo de alteración de la roca y medición de los cambios en ella asumiendo un modelo constitutivo de la roca.
- Las mediciones deben estar lejos de las excavaciones o alteraciones (> 1.5 – 2.0 veces el diámetro de la apertura)



Flatjack



K. Kim y J. A. Franklin (1987)

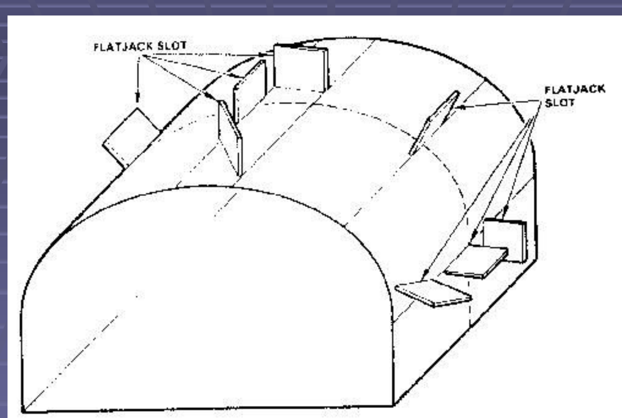


Fig. 2. Example of the layout of slots for flatjack tests.

K. Kim y J. A. Franklin (1987)

Mediciones de deformación

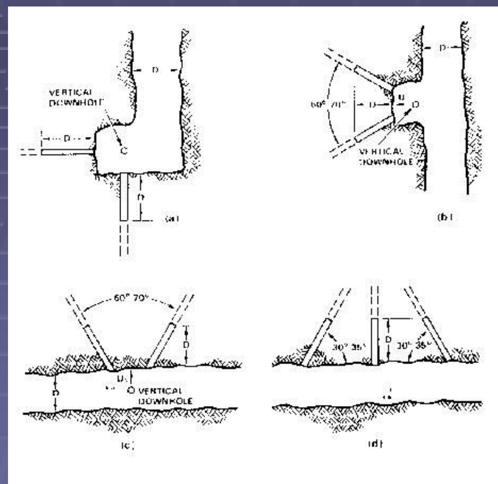
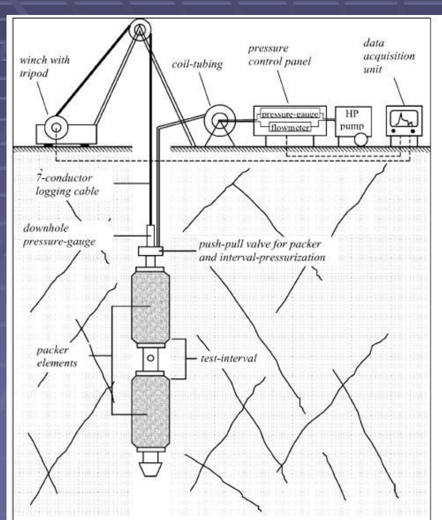
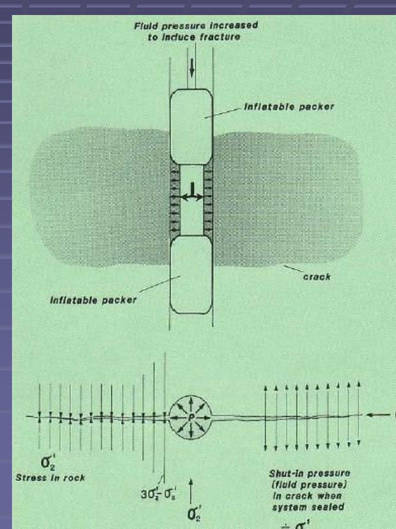


Fig. 7. Drillhole configuration.

Fracturación hidráulica





Fracturación hidráulica

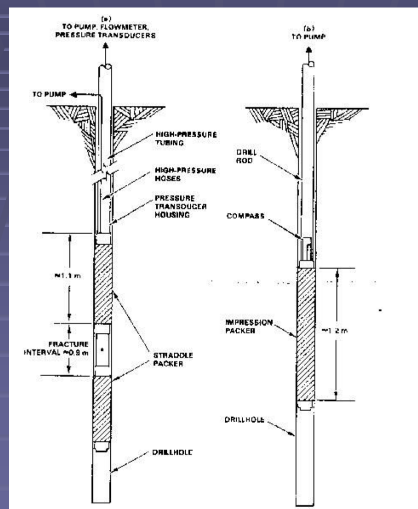


Fig. 4. Schematic representation of (a) hydrofracturing tool and (b) impression packer.

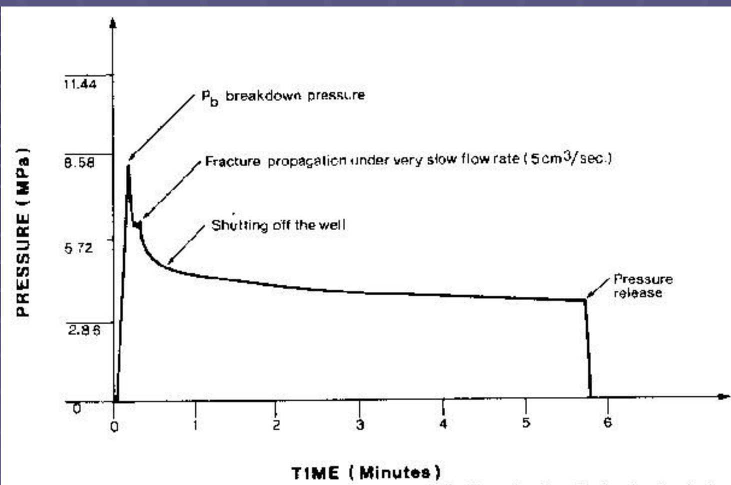
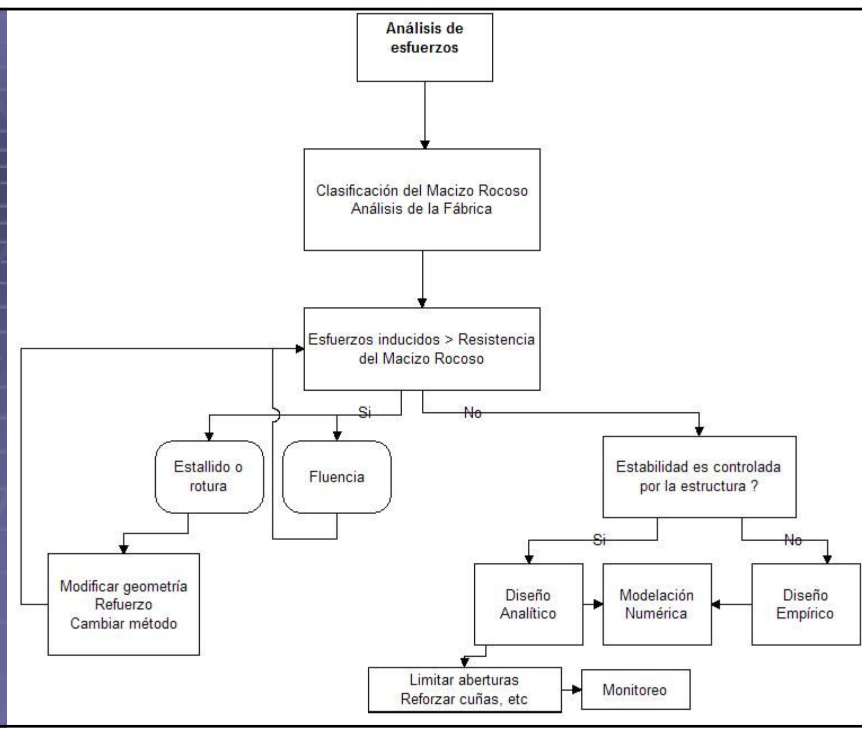
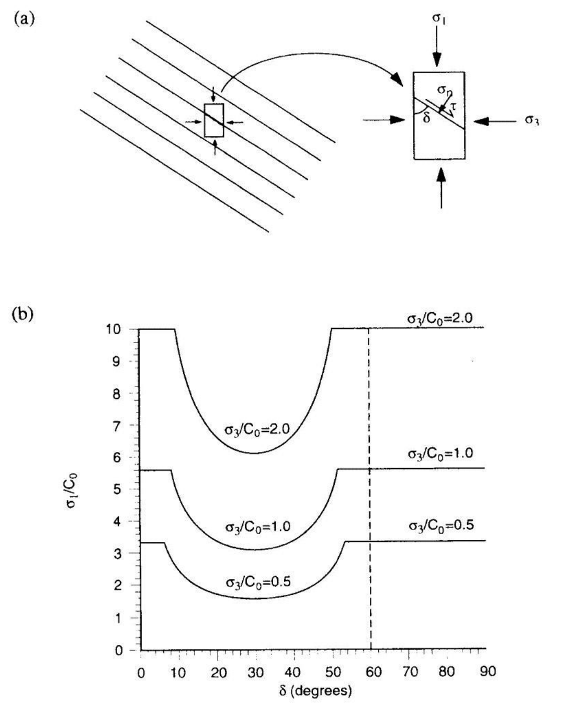


Fig. 1. Typical pressure-time record observed during a hydraulic fracturing test.

Propiedades de la roca intacta





Diferencia con la resistencia del macizo rocoso

Influencia de discontinuidades

Ejemplo:

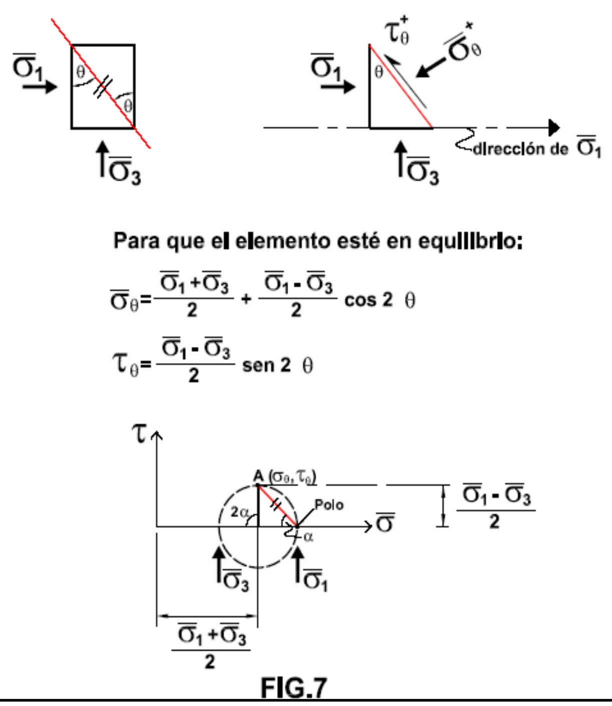
Roca intacta

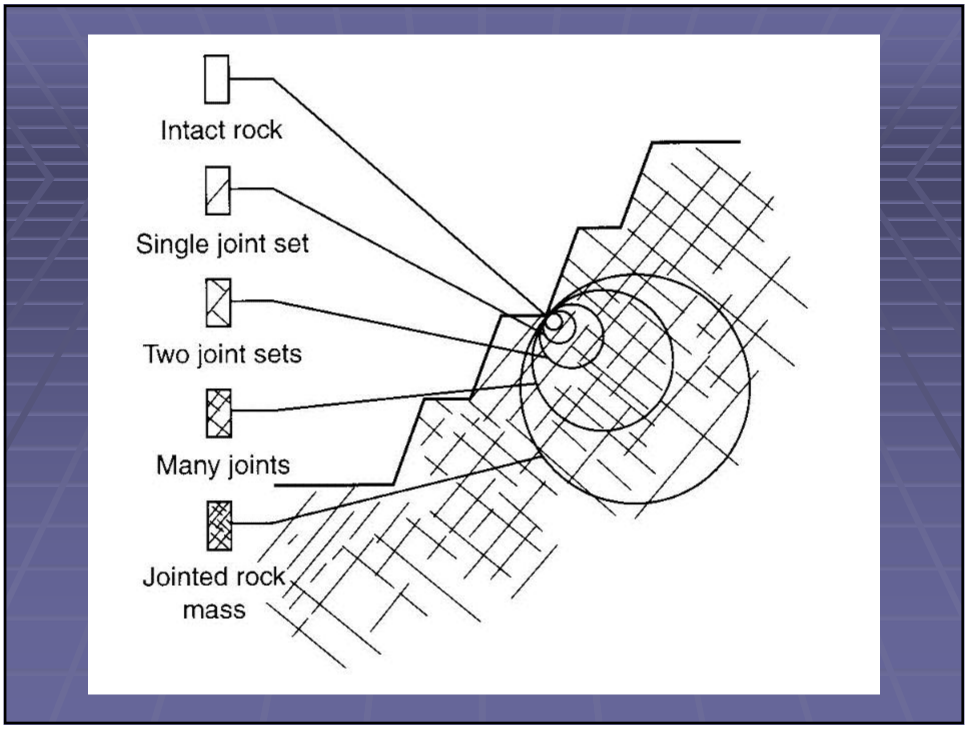
c = 5 MPa

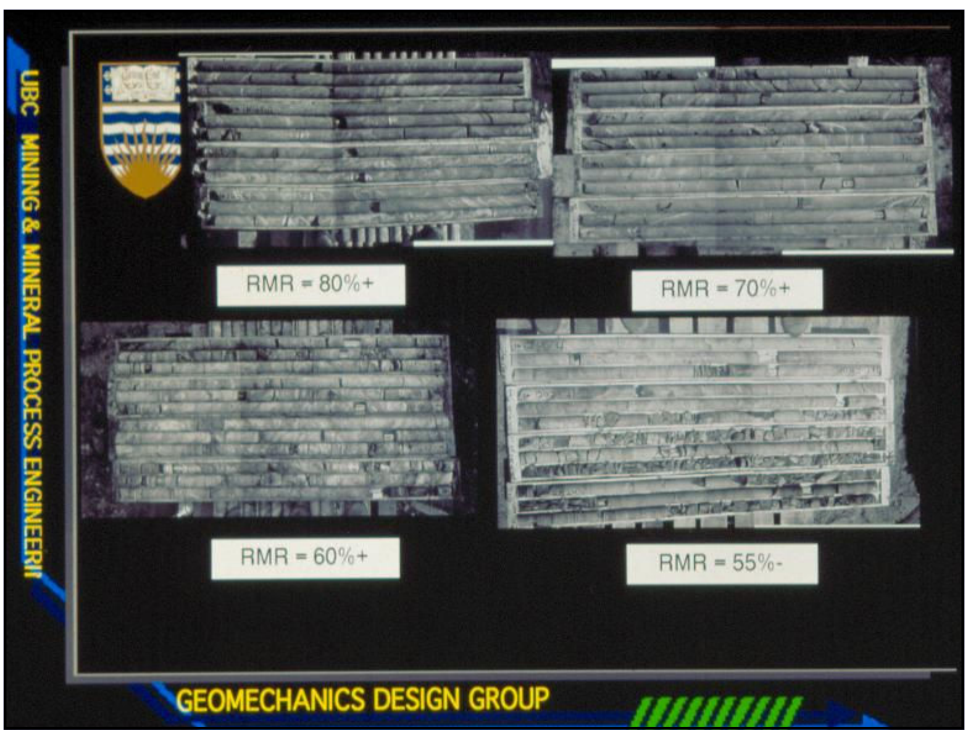
f = 40°

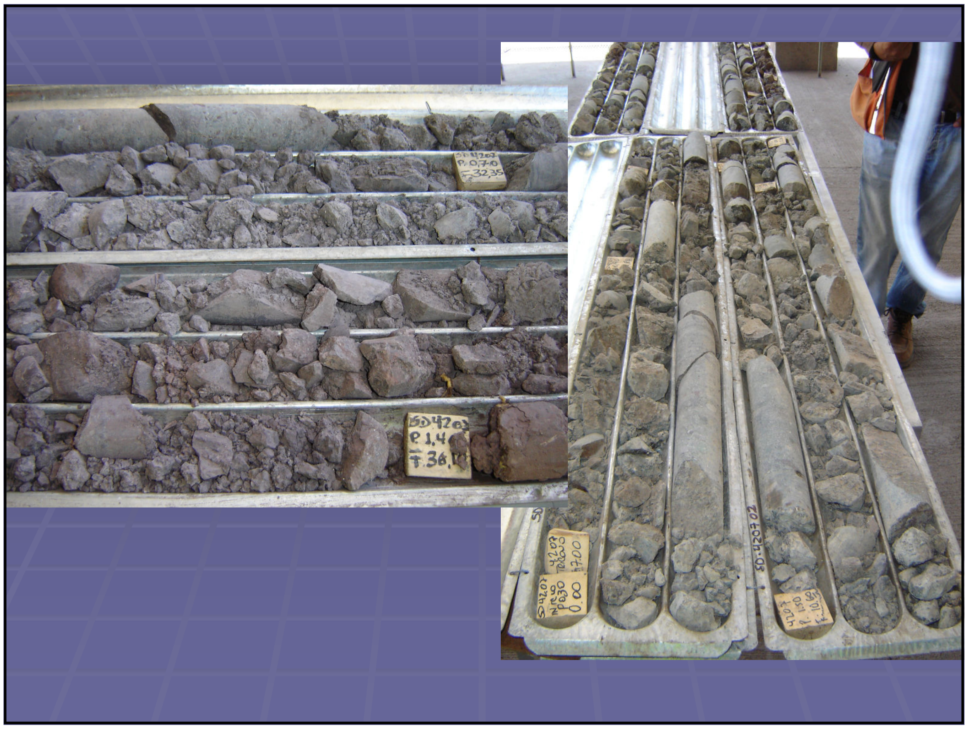
Discontinuidad

f = 30°




Intact rock
Single joint set
Two joint sets
Many joints
Jointed rock mass


UBC MINING & MINERAL PROCESS ENGINEERI
RMR = 80%+
RMR = 70%+
RMR = 60%+
RMR = 55%-
GEOMECHANICS DESIGN GROUP




G H
I J
K L

Existe mayor información sobre la roca intacta debido a la relativa facilidad de obtener muestras

Propiedades de la roca intacta

- Resistencia a la compresión no confinada
- Módulo de Young o de Deformación
- Módulo de Poisson
- Ángulo de fricción al deslizamiento
- Cohesión

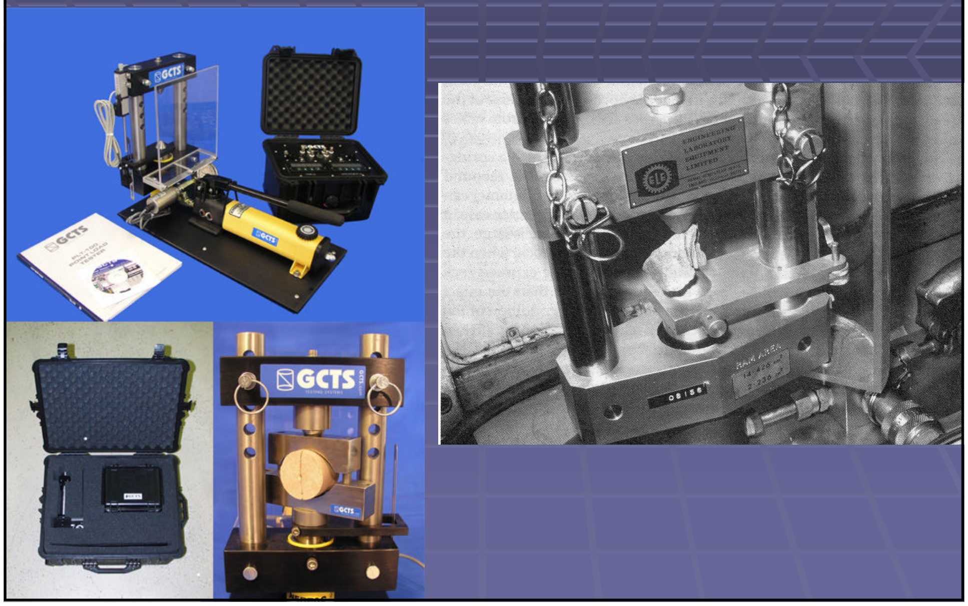
Propiedades índice

- Peso unitario, γ
- Porosidad, n
- Índice de durabilidad, ID
- Velocidad de propagación de ondas P, V_p
- Velocidad de propagación de ondas S, V_S

Tipos de resistencia

- Resistencia a la tracción directa, TS
- Resistencia en carga puntual, Is
- Resistencia en compresión uniaxial, UCS
- Resistencia en compresión triaxial

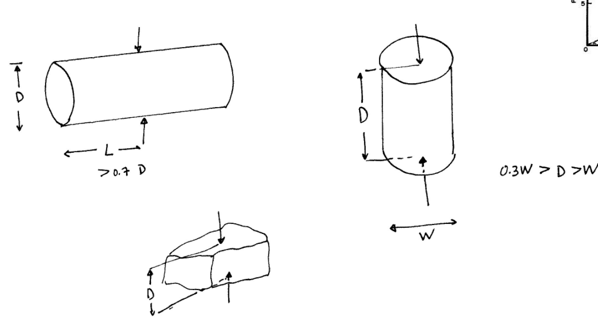
Ensayo de Carga Puntual

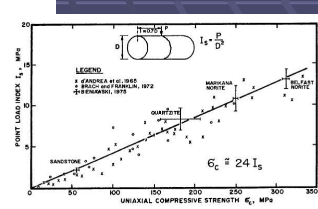


$I_s = \dfrac{P}{D^2}$ [unidades de esfuerzo]

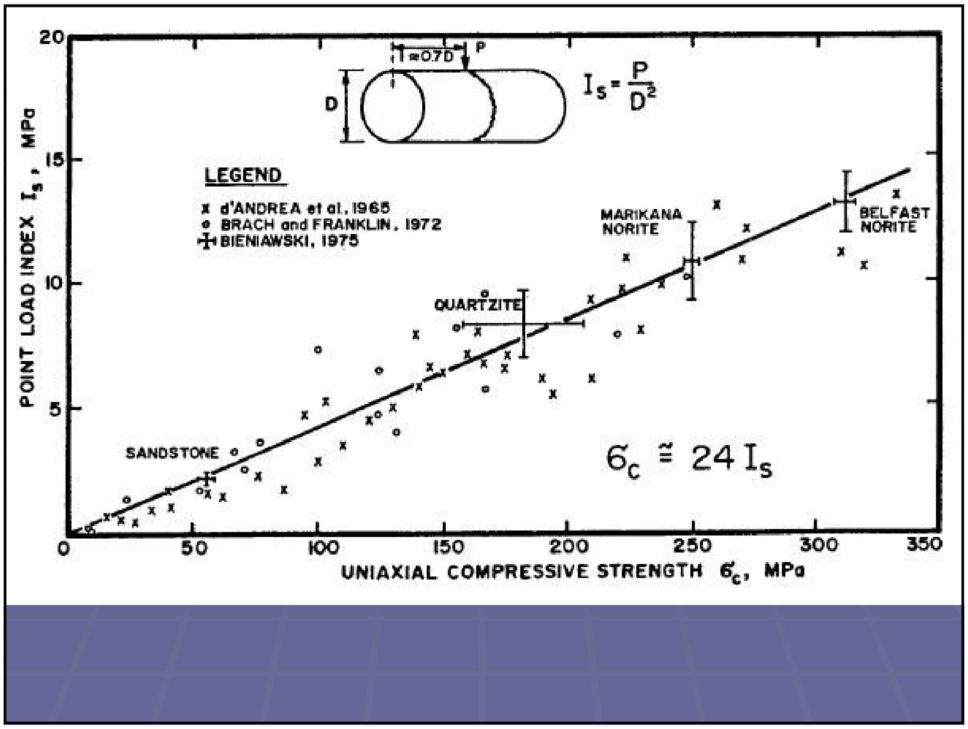
↳ Point load index (Franklin 1985)

I_{s50} → el ensayo standard es realizado en un testigo cilíndrico de 50 mm de diámetro

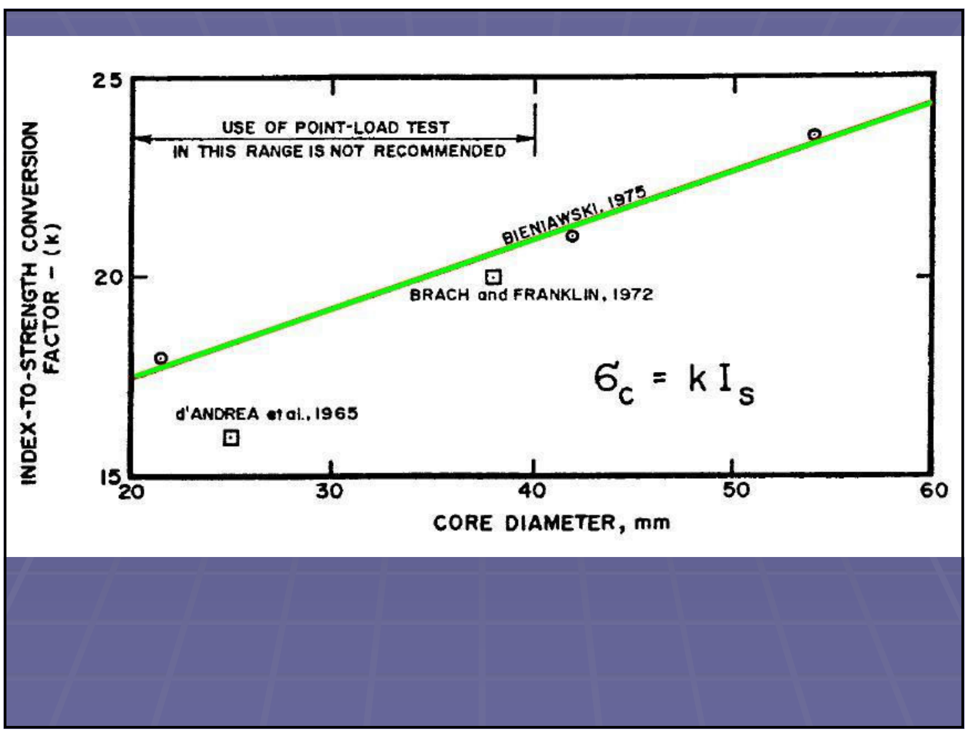




$D_e = \left(\dfrac{4A}{\pi}\right)^{0.5}$ A: área mínima del especimen



≈0.7D
P
D
$I_s = \frac{P}{D^2}$
LEGEND
x d'ANDREA et al. 1965
o BRACH and FRANKLIN, 1972
BIENIAWSKI, 1975
SANDSTONE
QUARTZITE
MARIKANA NORITE
BELFAST NORITE
$\sigma_c \cong 24 I_s$
POINT LOAD INDEX I_s, MPa
UNIAXIAL COMPRESSIVE STRENGTH σ_c, MPa



USE OF POINT-LOAD TEST
IN THIS RANGE IS NOT RECOMMENDED
BIENIAWSKI, 1975
BRACH and FRANKLIN, 1972
d'ANDREA et al., 1965
$\sigma_c = k I_s$
INDEX-TO-STRENGTH CONVERSION FACTOR – (k)
CORE DIAMETER, mm

Classification of rock and soil strengths (ISRM, 1981b)

Description	*Field identification*	*Approx. range of uniaxial compressive strength (MPa)*
Extremely strong rock	Specimen can only be chipped with geological hammer.	>250
Very strong rock	Specimen requires many blows of geological hammer to fracture it.	100–250
Strong rock	Specimen requires more than one blow of geological hammer to fracture it.	50–100
Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer.	25–50
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	5.0–25
Very weak rock	Crumbles under firm blows with point of geological hammer and can be peeled by a pocket knife.	1.0–5.0
Extremely weak rock	Indented by thumbnail.	0.25–1.0
Hard clay	Indented with difficulty by thumbnail.	>0.5
Very stiff clay	Readily indented by thumbnail.	0.25–0.5
Stiff clay	Readily indented by thumb but penetrated only with great difficulty.	0.1–0.25
Firm clay	Can be penetrated several inches by thumb with moderate effort.	0.05–0.1
Soft clay	Easily penetrated several inches by thumb.	0.025–0.05
Very soft clay	Easily penetrated several inches by fist.	<0.025

Duncan & Mah, 2004