109.01

Engineering Geology
工程地質

Ch3 岩石、岩體與工程(Part 3)

岩體的工程性質

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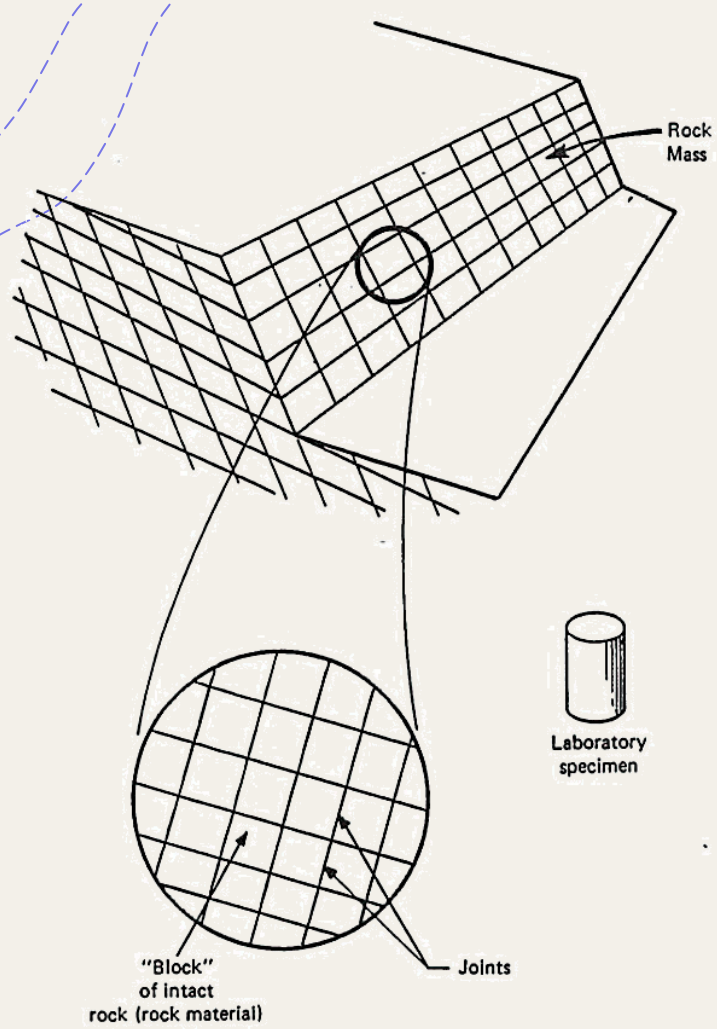
大綱

1. 岩體的概念
2. 岩石的強度、變形性及其相關特性
3. 岩石材料的力學性質
4. 不連續面(弱面)的力學性質
5. 岩體的力學性質

1.岩體的概念



Definitions of *rock mass* and *rock material*.

岩體
(rock mass)
力學性質
(強度、變形性、水力性質)
Mech. Properties
Strength, deformability, hydraulics

=

完整岩石
(intact rock)

+

不連續面
(discontinuity)

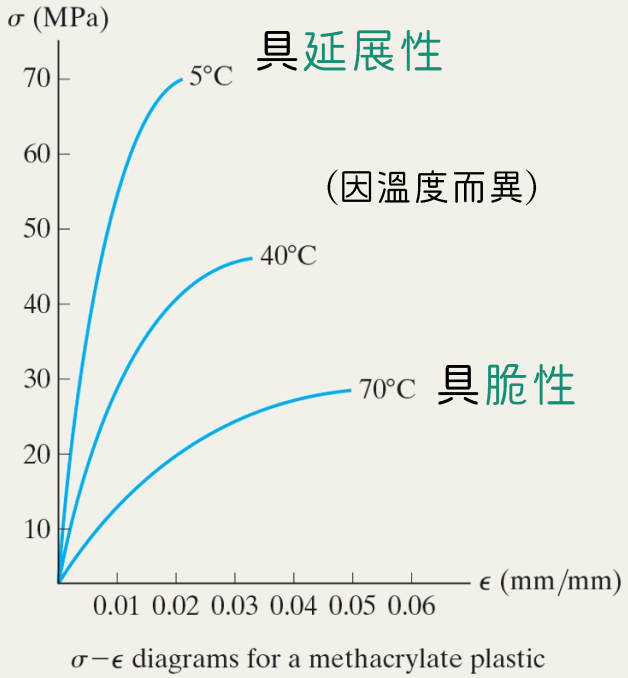
物質不連續
Discontinuity of the material

力學性質不連續
Discontinuity in Mechanics

2.岩石的強度、變形性及其相關特性

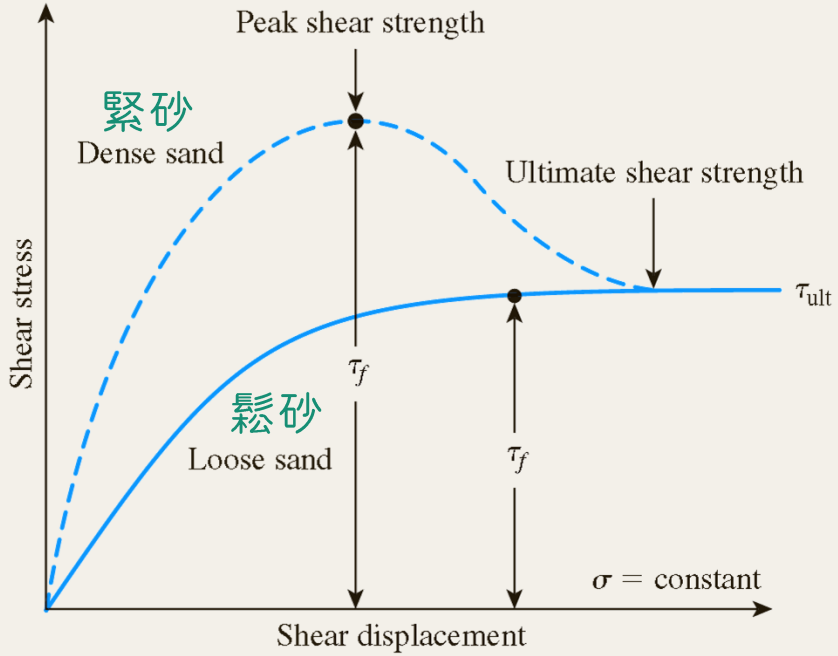
+ 室內試驗：直接剪力試驗、壓力試驗...
+ 應力 - 應變曲線
+ 延展性(ductile)材料：金屬
+ 脆性(brittle)材料：混凝土、土壤、岩石...

壓克力

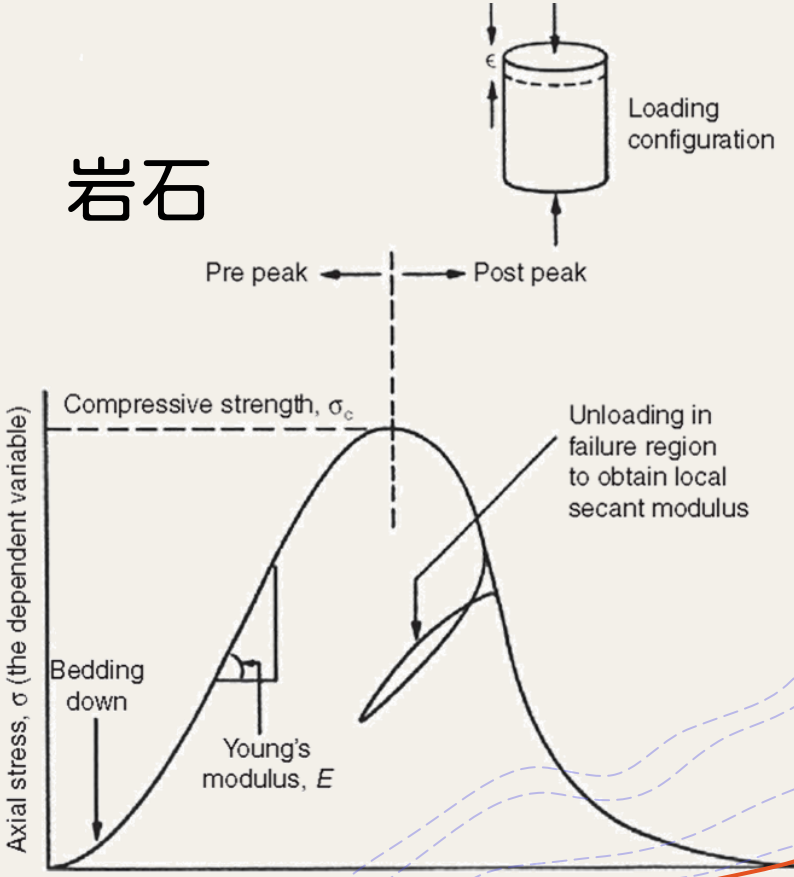


σ−ϵ diagrams for a methacrylate plastic

土壤

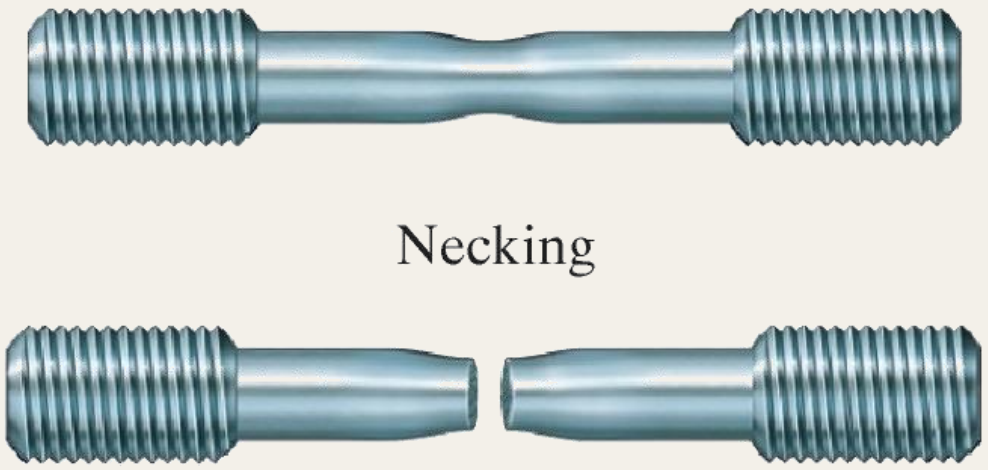


岩石



延展性材料
v.s.
脆性材料

金屬材料(受拉力破壞)

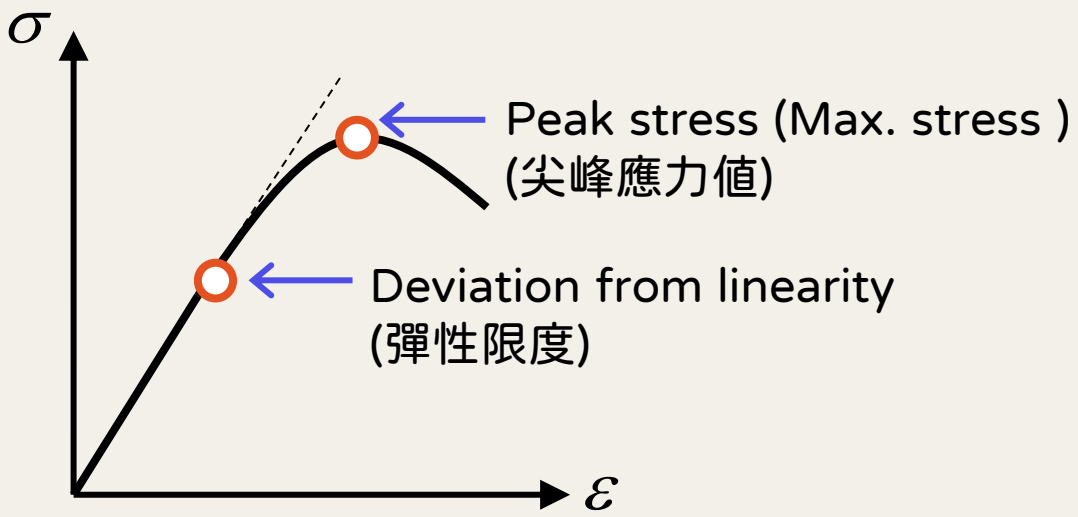


混凝土(受壓力破壞)

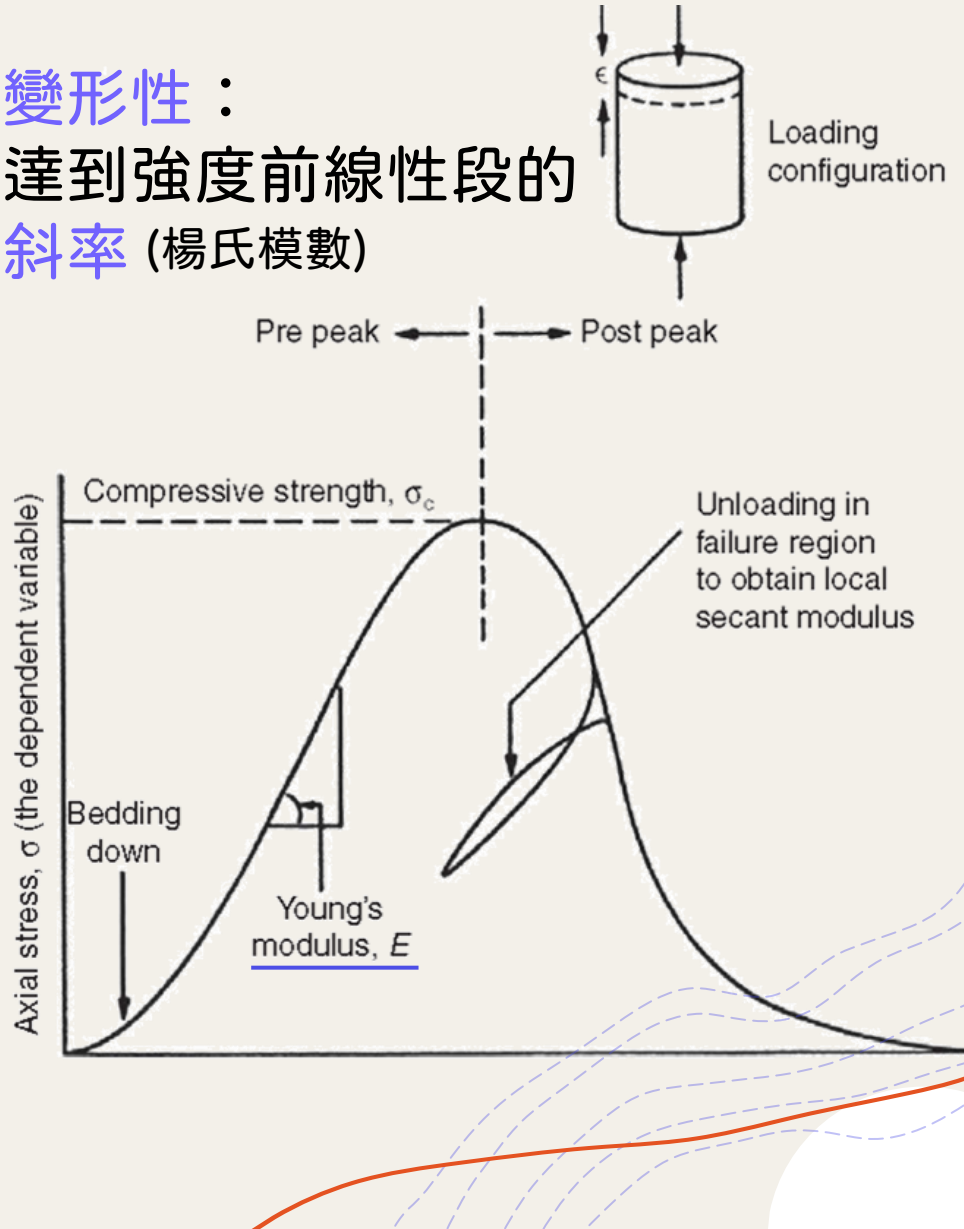
岩石的強度與變形性

岩石的破壞：
尖峰應力發生位置，稱為岩石的強度

Failure ≡ peak stress (Max. stress)
(strength)



變形性：
達到強度前線性段的
斜率 (楊氏模數)



岩石的楊氏模數如何決定？

%強度的切線斜率
(通常用50%)

線性段部分的
平均斜率

%強度的割線斜率
(通常用50%)

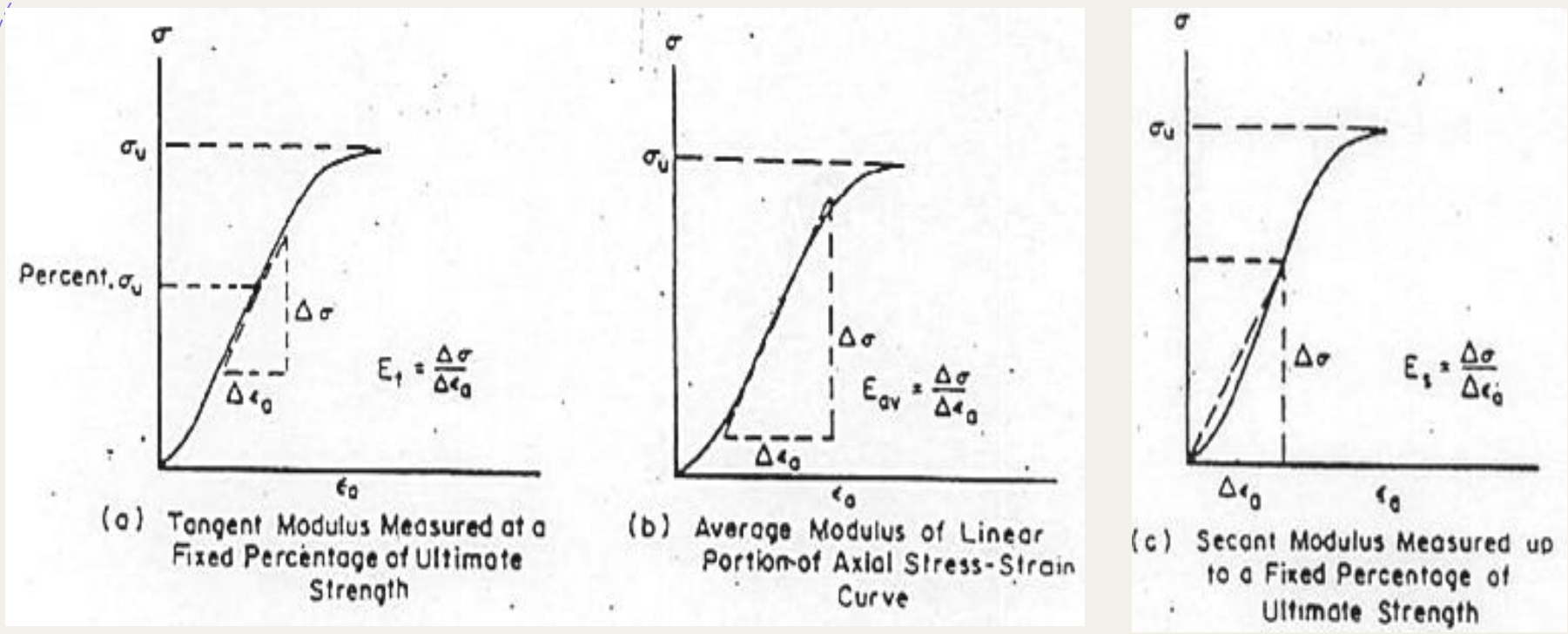


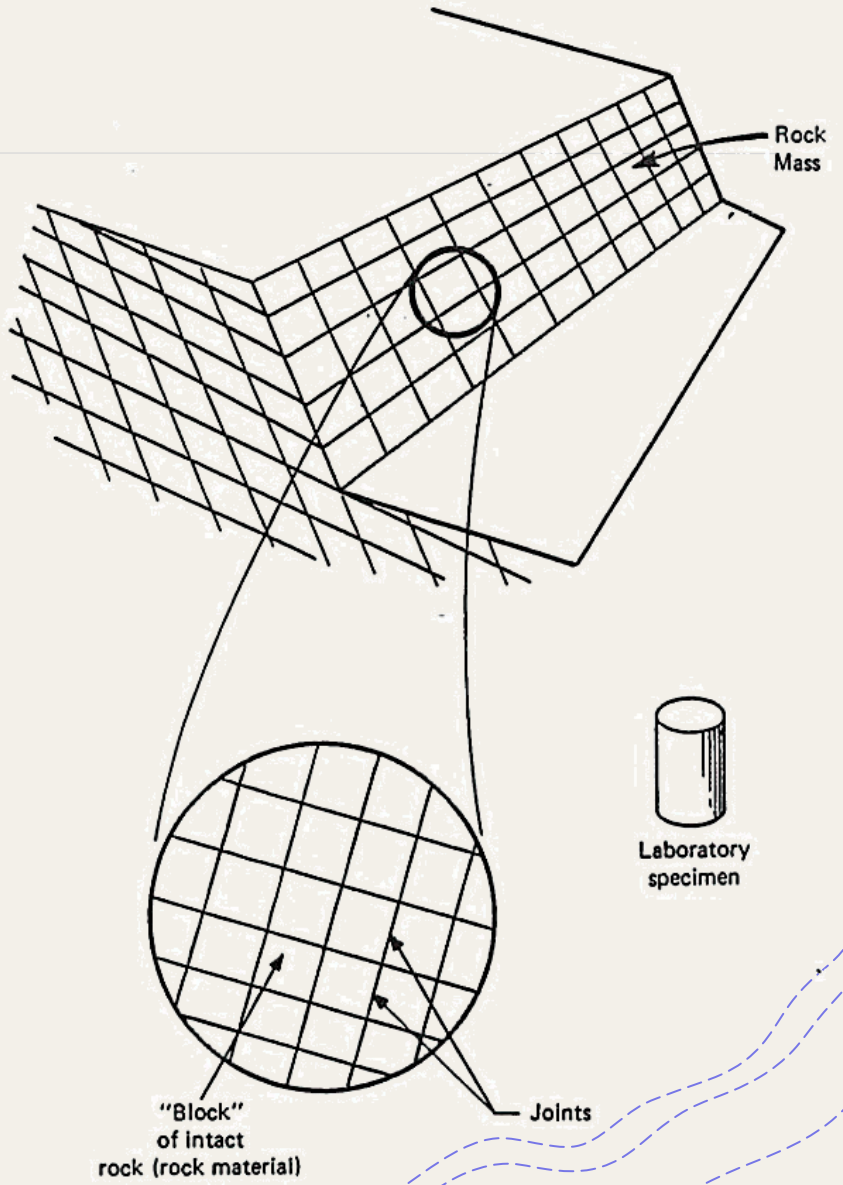
Fig. 2. Methods for calculating Young's modulus from axial stress-strain curve.

-Defined by ISRM(1981)

Scale effect (規模效應)

岩體(rock mass) → $n = \dfrac{\text{joints spacing } (s)}{\text{range of influence } (c)}$

	岩體	n
1	完整岩石材料 intact rock	10 ~ 1
2	部份岩石材料 intact rock ~ jointed rocks	1
3	A 開裂岩體 jointed rocks	1/10
	B 開裂岩體 Highly jointed rocks	1/100
4	高度破碎岩體 (土、堆石) gravels?	1/1000



Definitions of *rock mass* and *rock material*.

3.岩石材料的力學性質

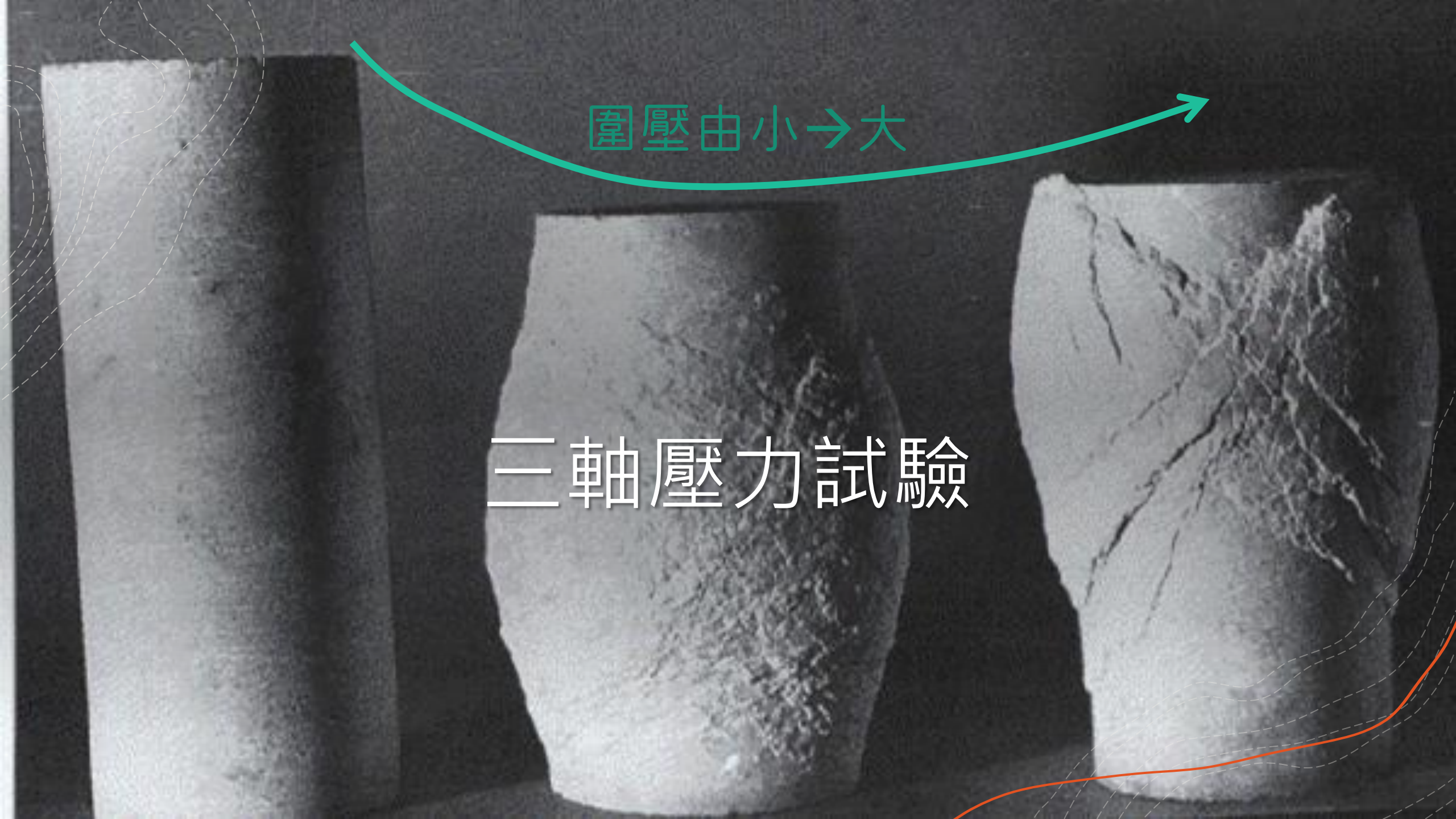
+指數特性：孔隙率、密度、透水性(滲透性)、強度、消散耐久指數、波速

+室內力學試驗：張力強度試驗、壓力強度試驗、剪力強度試驗

+岩石材料受壓的強度與變形性

+岩石材料的破壞準則：Mohr-Coulomb, Hoek-Brown

+現地試驗



三軸壓力試驗

增加圍壓可使岩石由脆性轉變為延展性

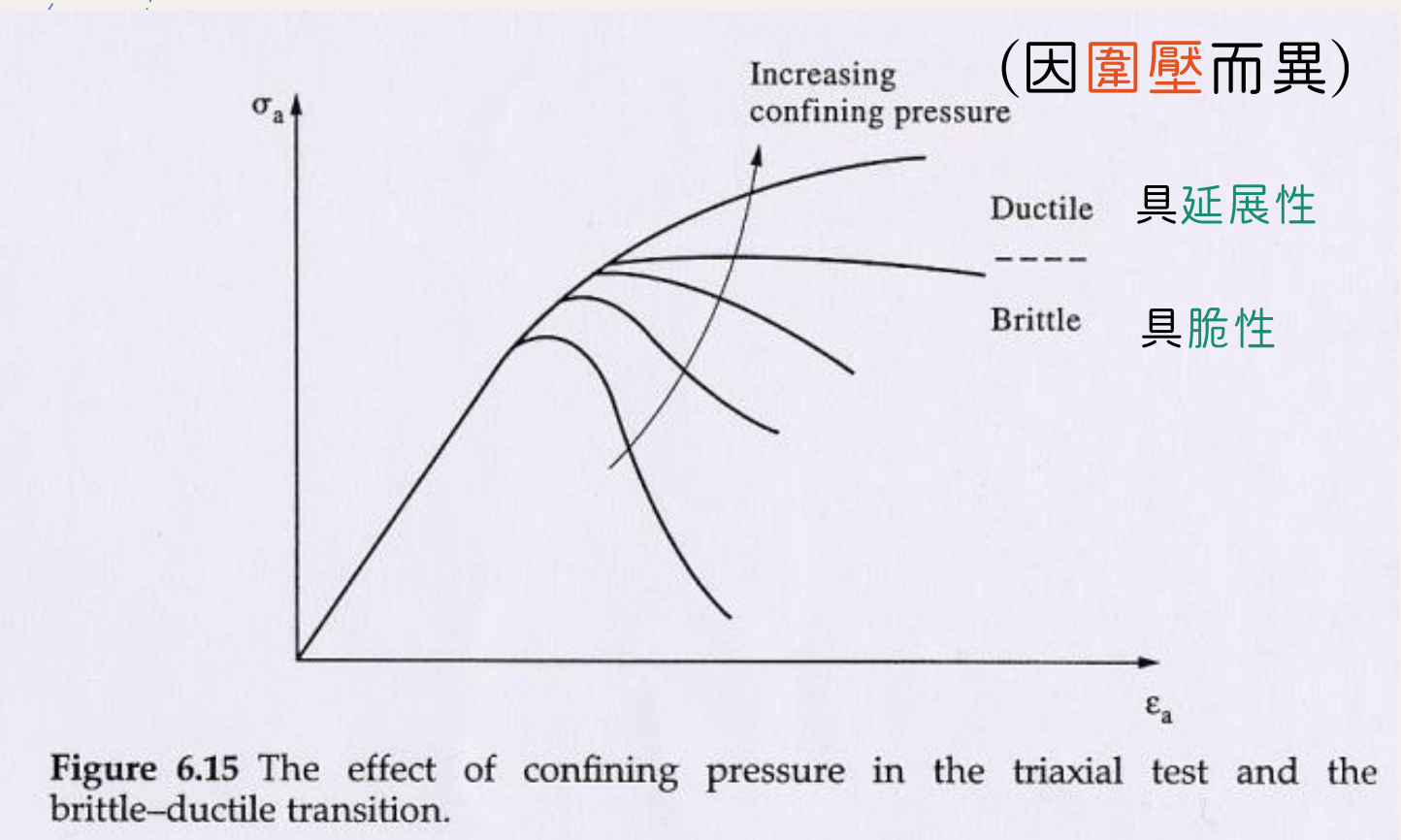
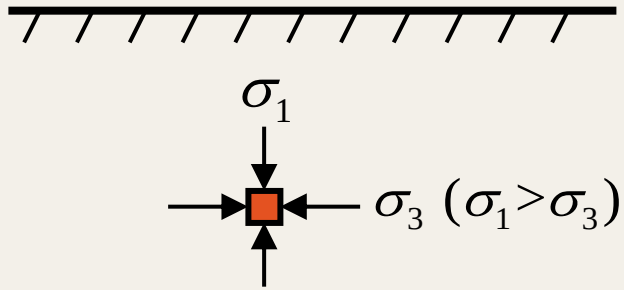


Figure 6.15 The effect of confining pressure in the triaxial test and the brittle–ductile transition.

圍壓？



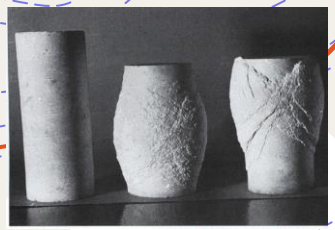
圍壓p增加 ↗

(1) Brittle→Ductile

(2) Deformability ↑

(3) Strength ↑

(4) Failure pattern



Mohr - Coulomb Failure Criterion

(1) τ-σ plane

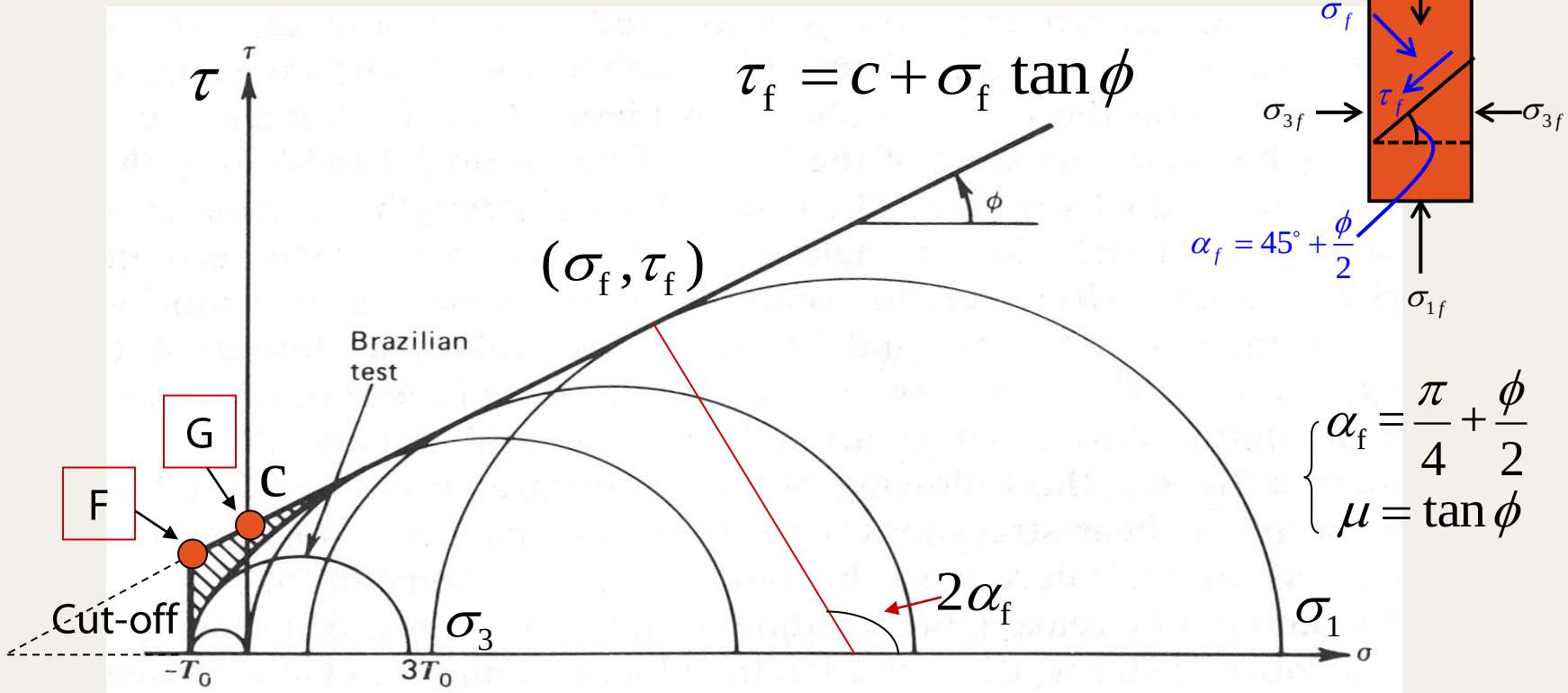
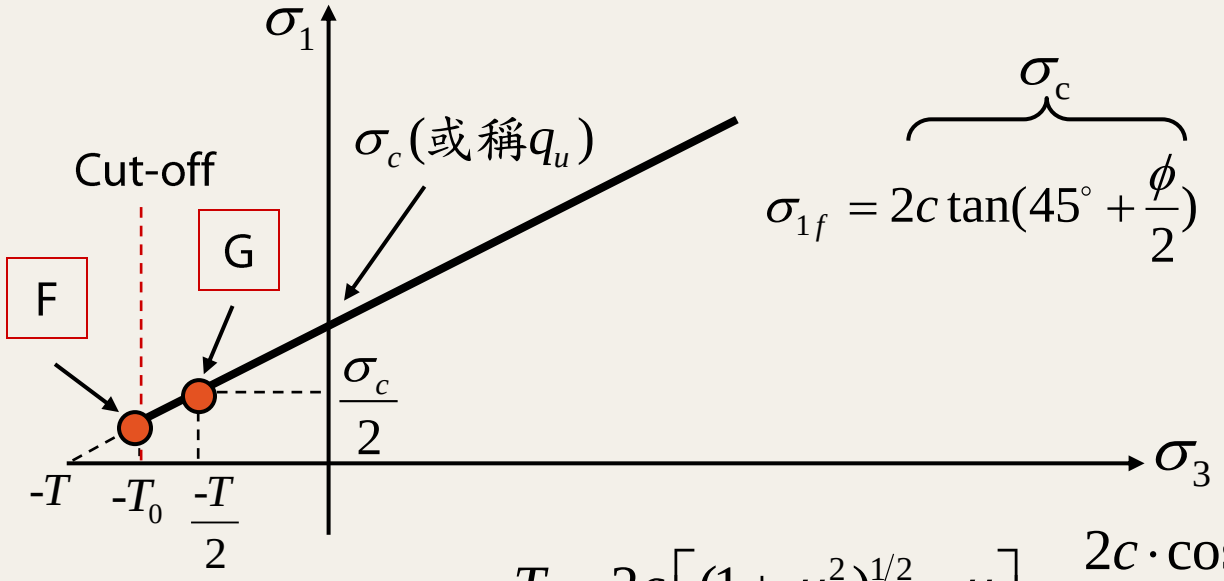


Figure 3.16 Comparison of empirical envelope and Mohr-Coulomb criterion in the tensile region. Inside the ruled region, the Mohr-Coulomb criterion with tension cutoff overestimates the strength.

Mohr - Coulomb Failure Criterion

(2) σ_1-σ_3 plane



T_0: tensile strength

$$\sigma_{1f} = 2c\tan(45^\circ + \frac{\phi}{2}) + \sigma_{3f}\tan^2(45^\circ + \frac{\phi}{2})$$

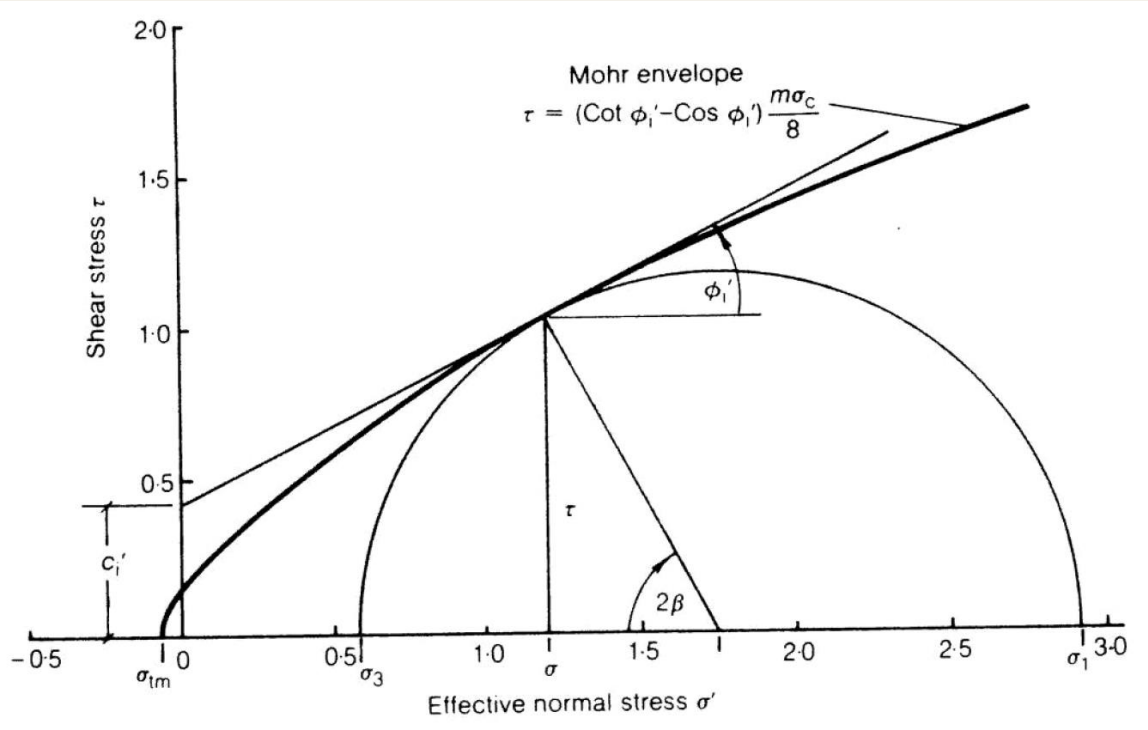
(σ_c = $2c\tan(45^\circ + \frac{\phi}{2})$)

$$T = 2c\left[(1+\mu^2)^{1/2} - \mu\right] = \frac{2c\cdot\cos\phi}{1+\sin\phi} \quad \text{(apparent)}$$

$$\sigma_c = 2c\left[(1+\mu^2)^{1/2} + \mu\right] = \frac{2c\cdot\cos\phi}{1-\sin\phi}$$

Hoek - Brown Failure Criterion(1980)

Hoek and Brown (1980a)



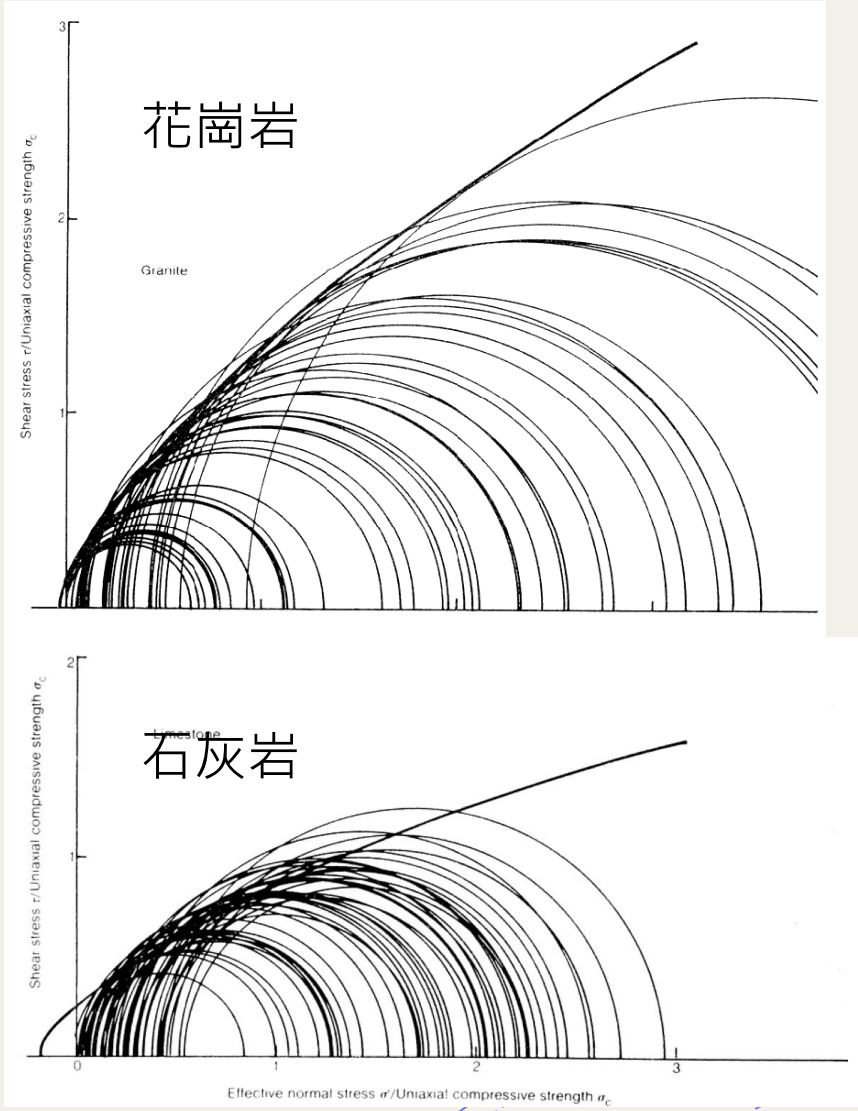


Figure 6 : Mohr failure circles for published triaxial test data for intact samples of (a) granite and (b) limestone.

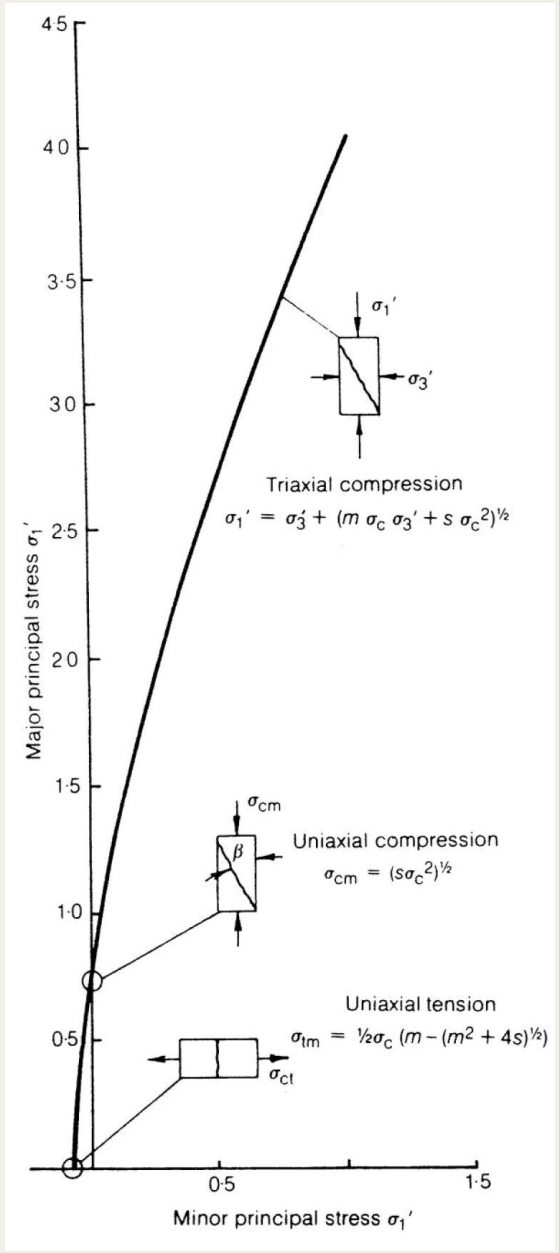
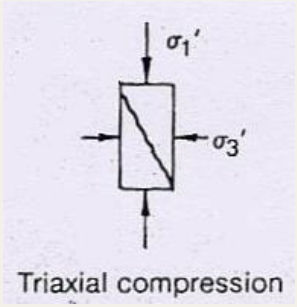


Figure 3. Summary of equations with the non-linear failure criterion proposed by Hoek & Brown (1980b)

Hoek - Brown Failure Criterion(1980)

(1) σ_1-σ_3 plane



$$\sigma_1' = \sigma_3' + (m\sigma_c\sigma_3' + s\sigma_c^2)^{1/2}$$

m: 完整岩材內顆粒大小 (ϕ有關)

s: 岩體破碎程度 (c有關)

4.不連續面的力學性質

1. 不連續面的描述與評估：位態、間距、持續性、粗糙度、內壁材料強度、內寬、軟弱夾心、滲水情形、組數、弱面密度與岩體幾何形狀
2. 不連續面的室內力學試驗：剪力強度試驗(正向、剪向)
3. 不連續面的強度與變形性
4. 不連續面的破壞準則：Mohr-Coulomb, Barton

4.1不連續面的描述與評估

參考**第二章**

+ 位置、方位
+ 間距
+ 持續性
+ 粗糙度
+ 內壁材料強度
+ 內寬
+ 軟弱夾心
+ 滲水情形
+ 組數
+ 弱面密度與岩體幾何形狀

4.2不連續面的室內力學試驗

Li et al. (2016)　岩石節理試體照片

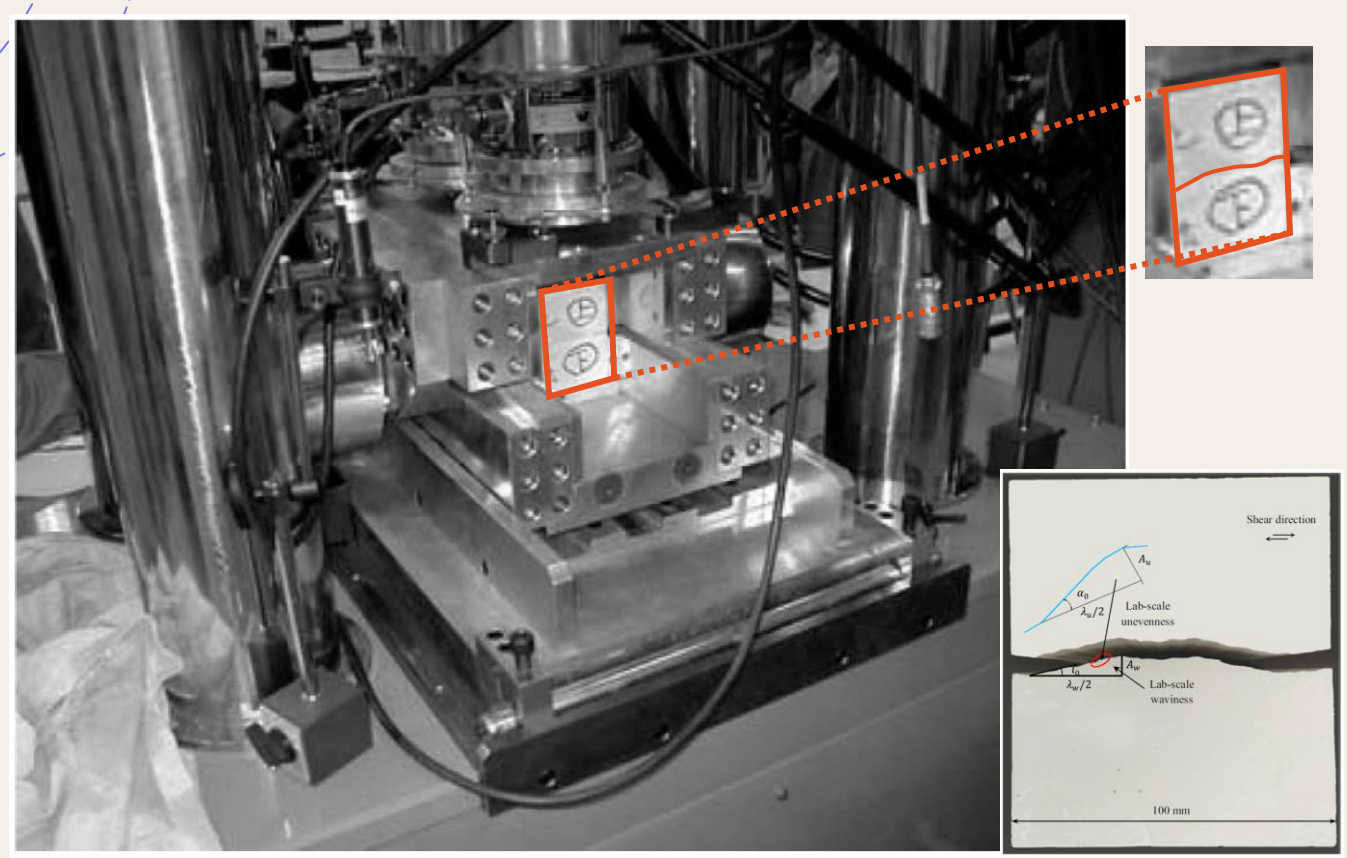


Photo 2 Shear test box.

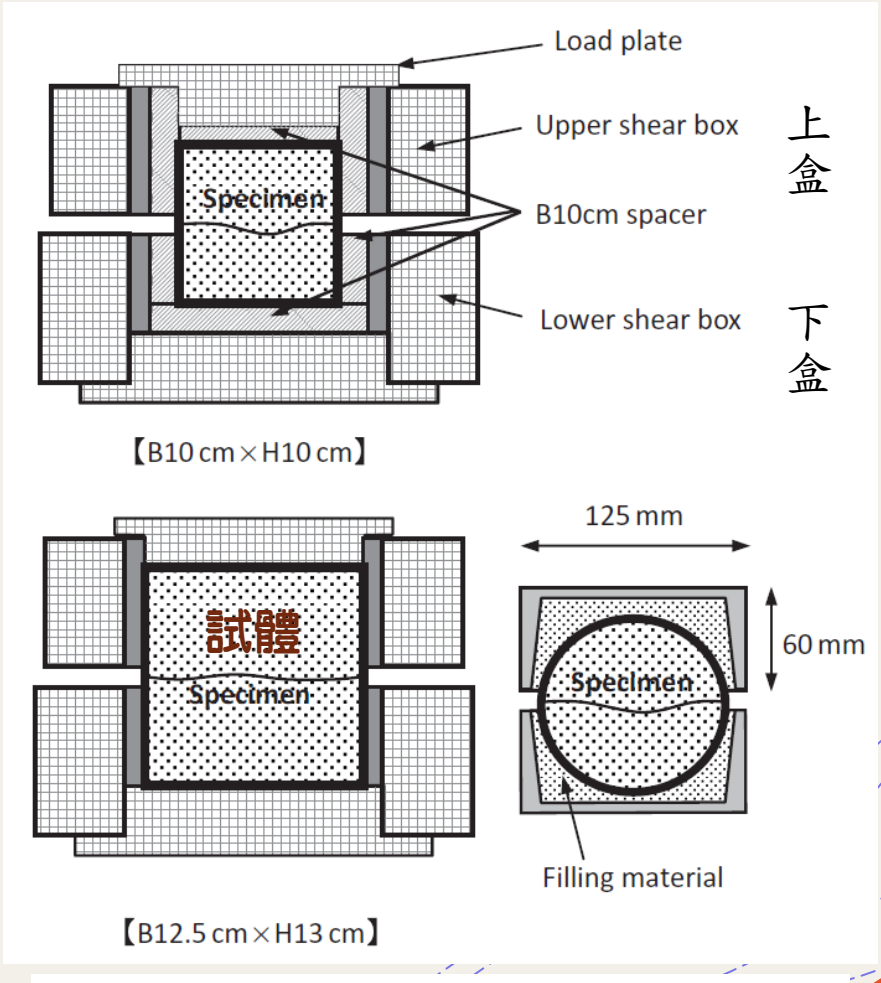
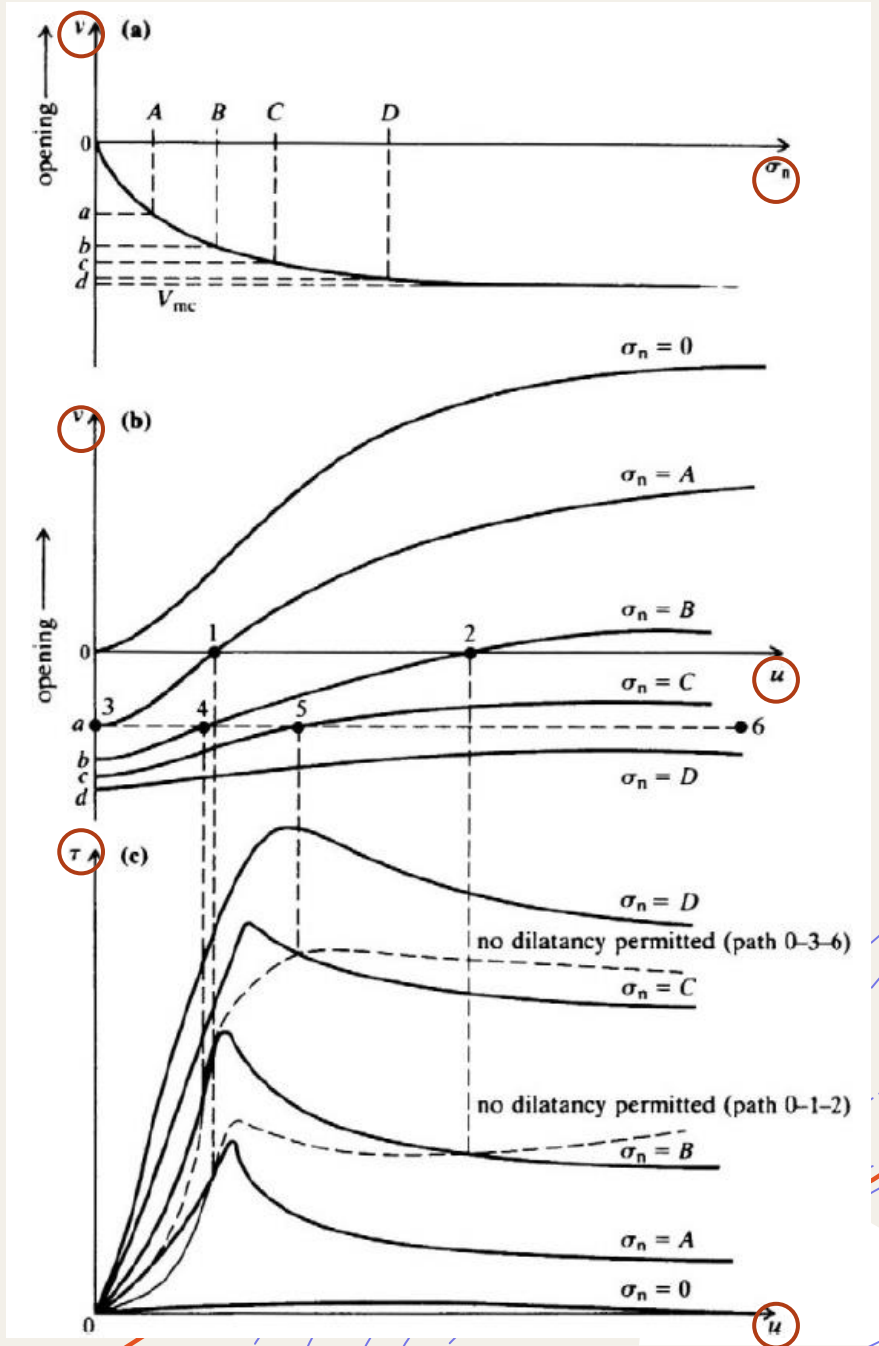


Figure 3 Structure of shear box and preparation of artificial joint sample.

Jiang, Y. J. (2016). Rock joints shearing testing system in X.T . Feng (Ed.), *Rock Mechanics and Engineering* Vol. 2 (pp. 217-249). London, UK: Taylor & Francis Group.

4.3 不連續面的強度與變形性

+ **正向應力&正向位移**
  + 閉合曲線
  + σ & Δv 關係
+ **剪應力&剪位移**
  + 剪力變形曲線
  + τ & Δu 關係
+ **剪位移&正向位移**
  + 膨脹曲線(剪動時垂直向位移的變化)
  + Δu & Δv 關係



不連續面的剪力強度

1. 尖峰摩擦角, ϕ_p
2. 殘餘摩擦角, ϕ_r
3. 基本摩擦角, ϕ_b (basic friction angle)

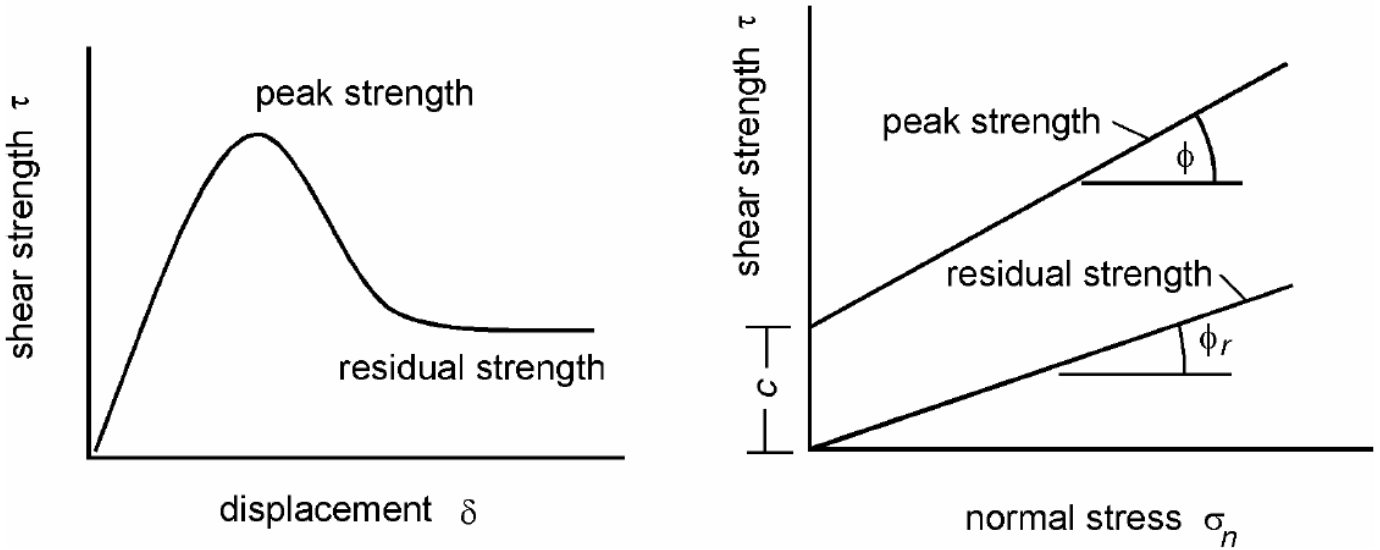
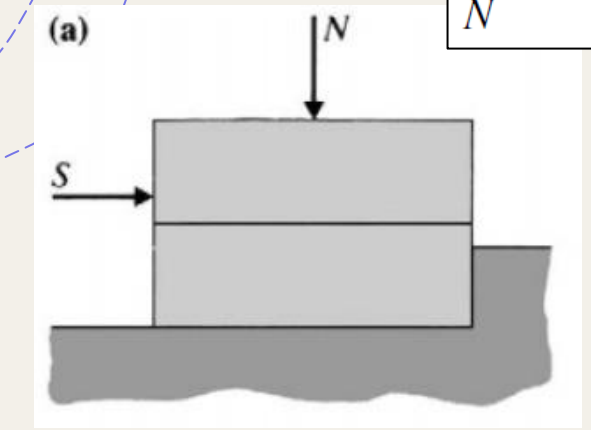


Figure 1: Shear testing of discontinuities

表面粗糙度對剪力強度的影響

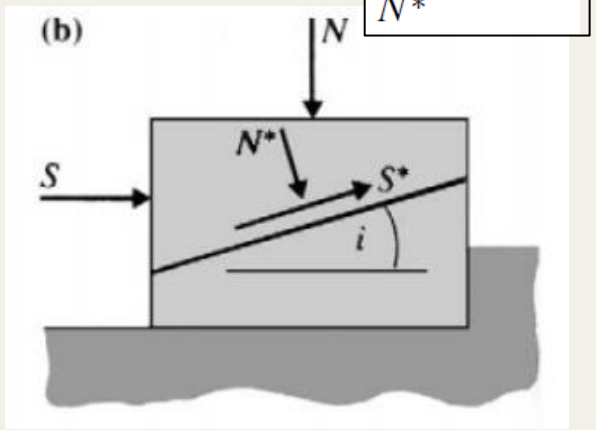
平滑弱面

$$\frac{S}{N} = \tan \phi$$



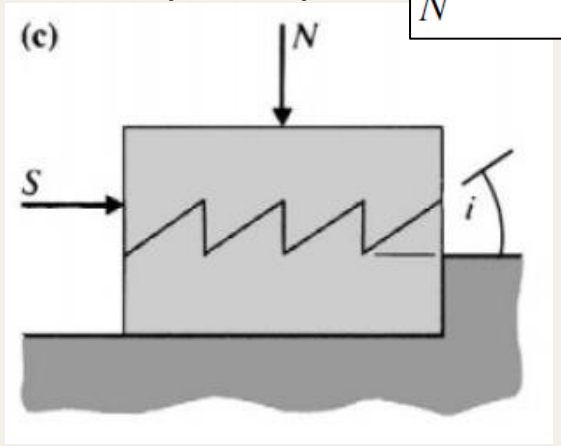
平滑弱面+傾斜

$$\frac{S^*}{N^*} = \tan \phi$$



規則起伏(鋸齒狀)弱面

$$\frac{S}{N} = \tan (\phi + i)$$



$$\frac{S}{N} = \tan (\phi + i)$$

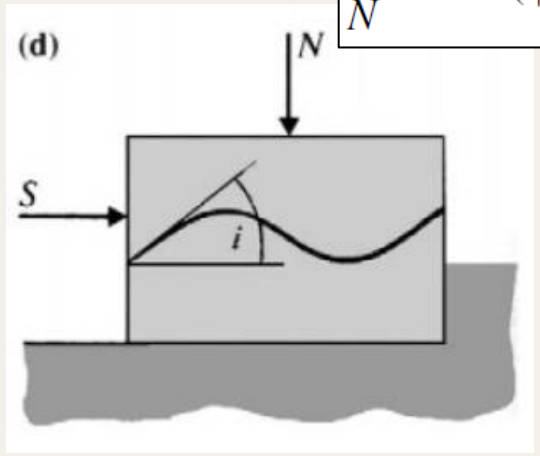


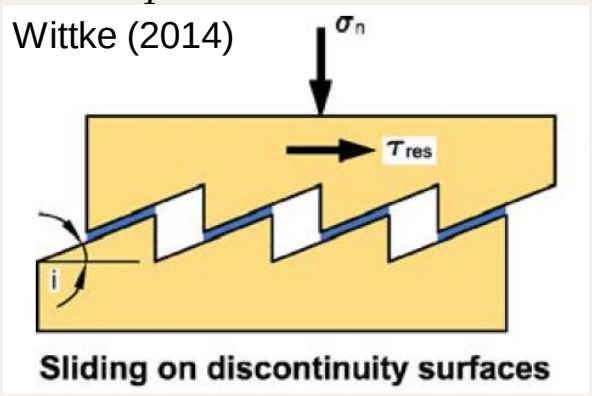
Figure 4.41 Idealised surface roughness models illustrating the roughness angle, i.

Brady and Brown (2005)

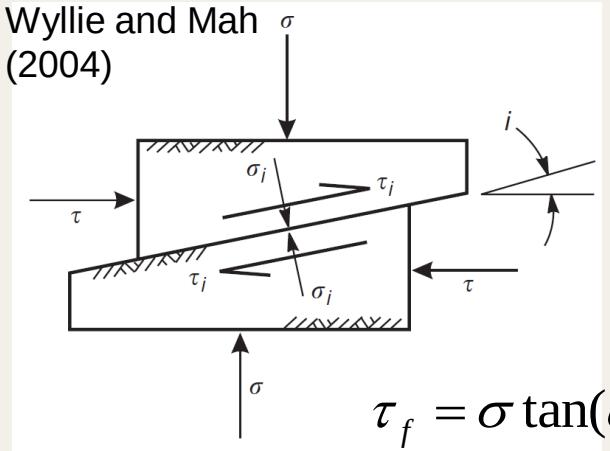
不連續面之力學行為:(1)爬坡；(2)剪斷

+(1) $\frac{\sigma}{\sigma_T} << 1$ 爬坡 (sliding up)

Wittke (2014)



Wyllie and Mah (2004)

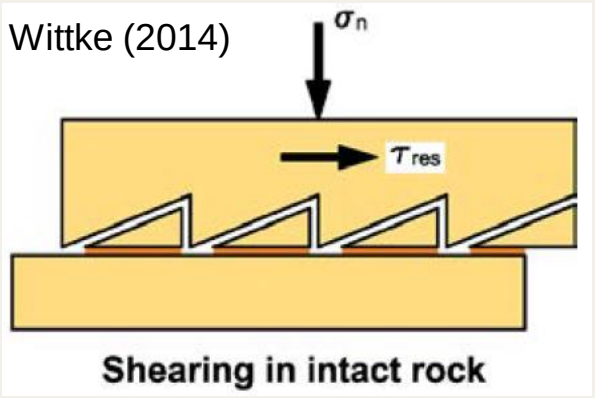


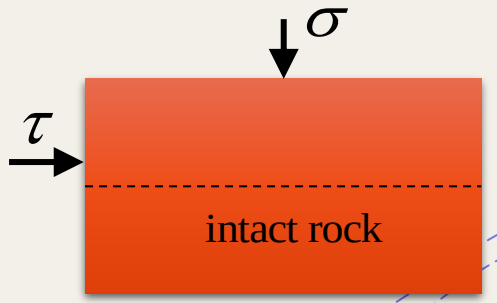
$\approx$

$\tau_f = \sigma \tan(\phi_b + i)$

+(2) $\frac{\sigma}{\sigma_T} \approx 1$ 剪斷 (shearing)

Wittke (2014)





$\tau_f = c_0 + \sigma \tan \phi_0$ c_0、ϕ_0: intact rock

4.4不連續面的破壞準則

+Patton's Bilinear model (1966)

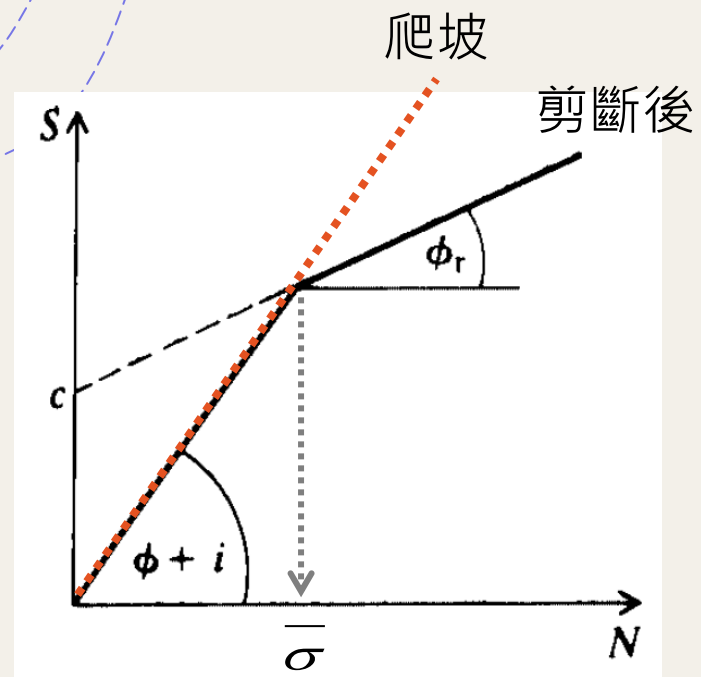
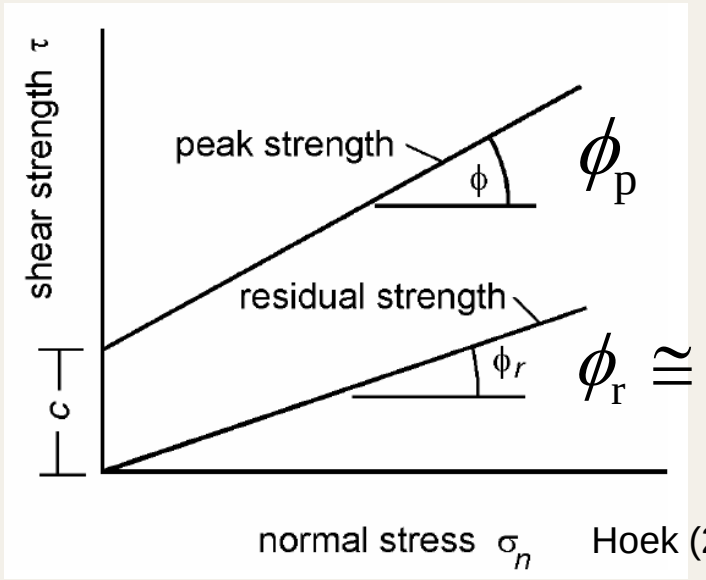


Figure 4.42 Bilinear peak strength envelope obtained in direct shear tests on the models shown in Figure 4.41.

Brady and Brown (2005)



Hoek (2007)

$\overline{\sigma}$: 轉換應力

$\sigma < \overline{\sigma}$: joint 爬坡 $\rightarrow \phi_p$

$\sigma > \overline{\sigma}$: intact rock破壞 $\rightarrow \phi_r \cong \phi_b$

4.4不連續面的破壞準則

B-B model

參數

$$\tau = \sigma_n \tan\left[JRC \log\left(\frac{JCS}{\sigma_n}\right) + \Phi_r\right]$$

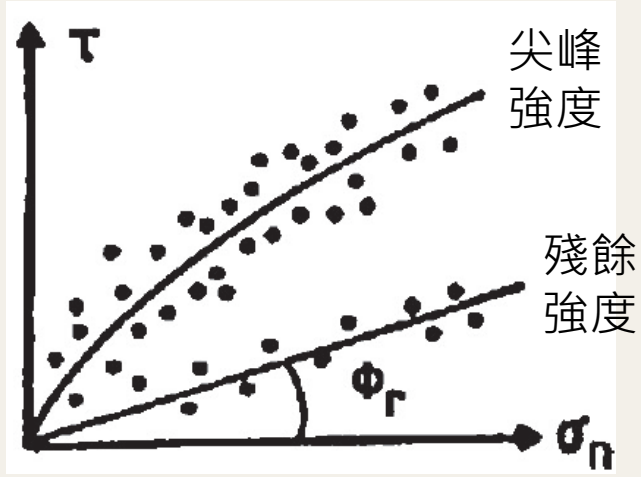
節理粗糙(度)係數

JRC = joint roughness coefficient

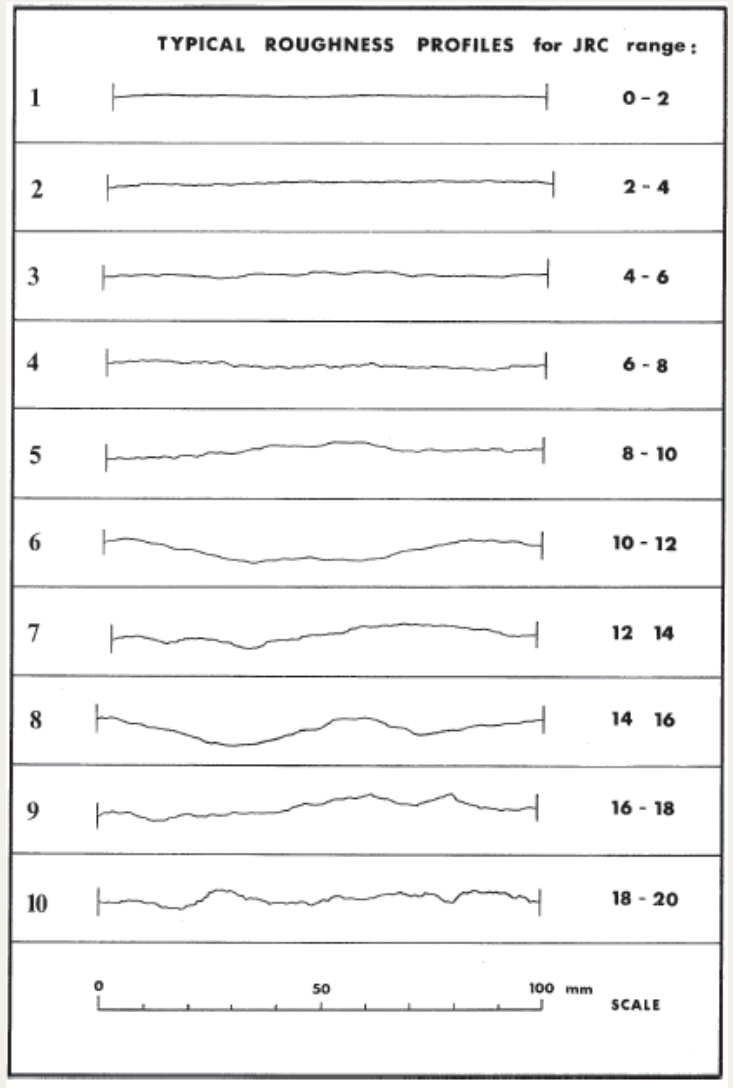
單軸壓縮強度

JCS = joint wall compression strength

Φ_r = residual friction angle

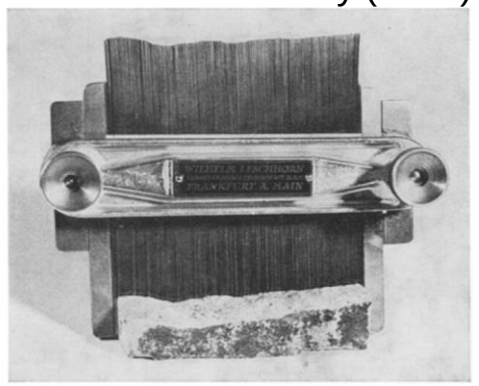


Barton (2013)

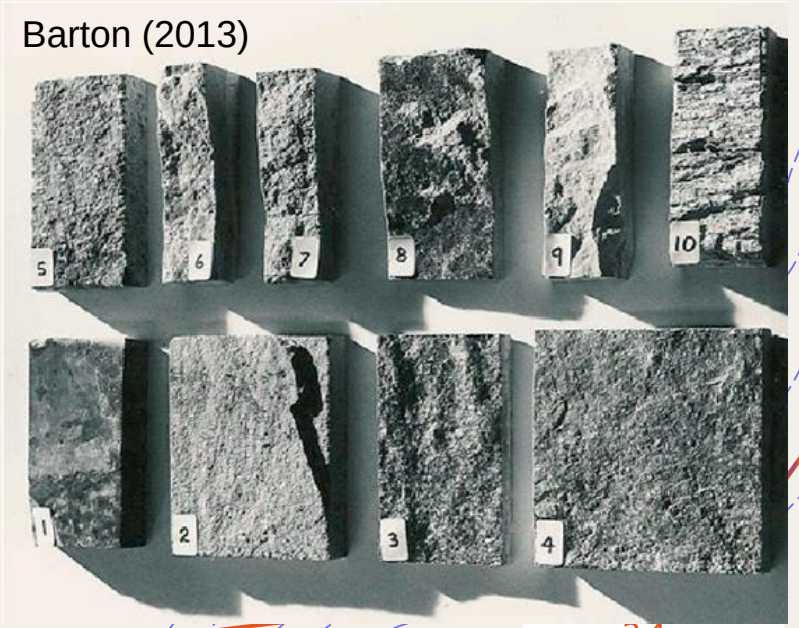


Barton and Choubey (1977)

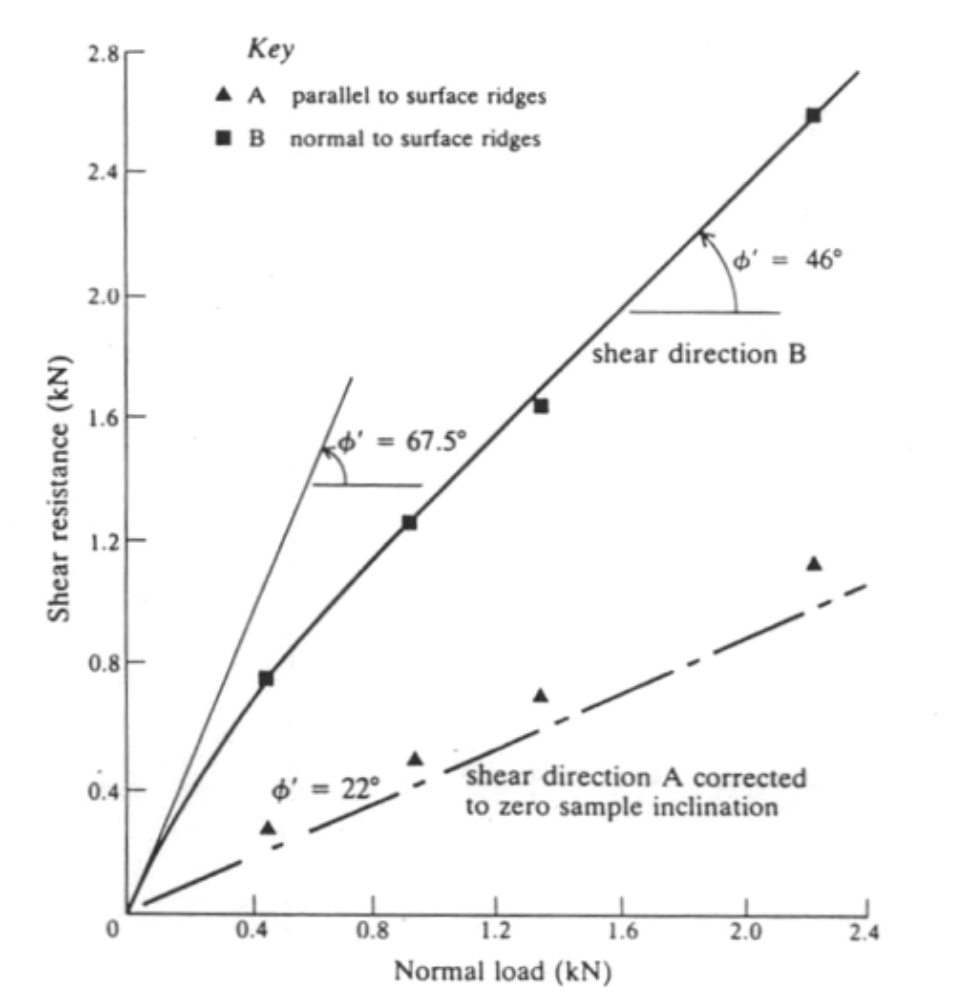
Barton and Choubey (1977)



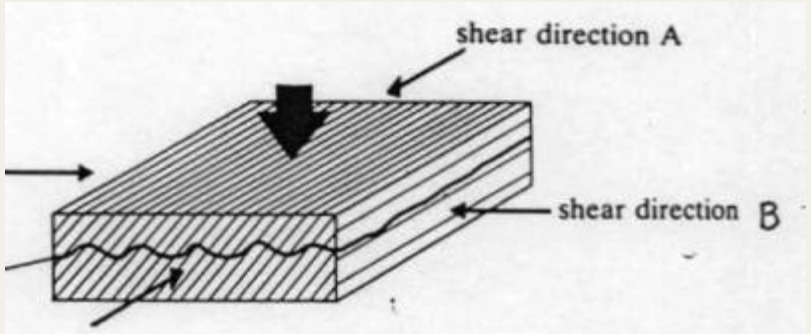
Barton (2013)



不連續面強度之異向性

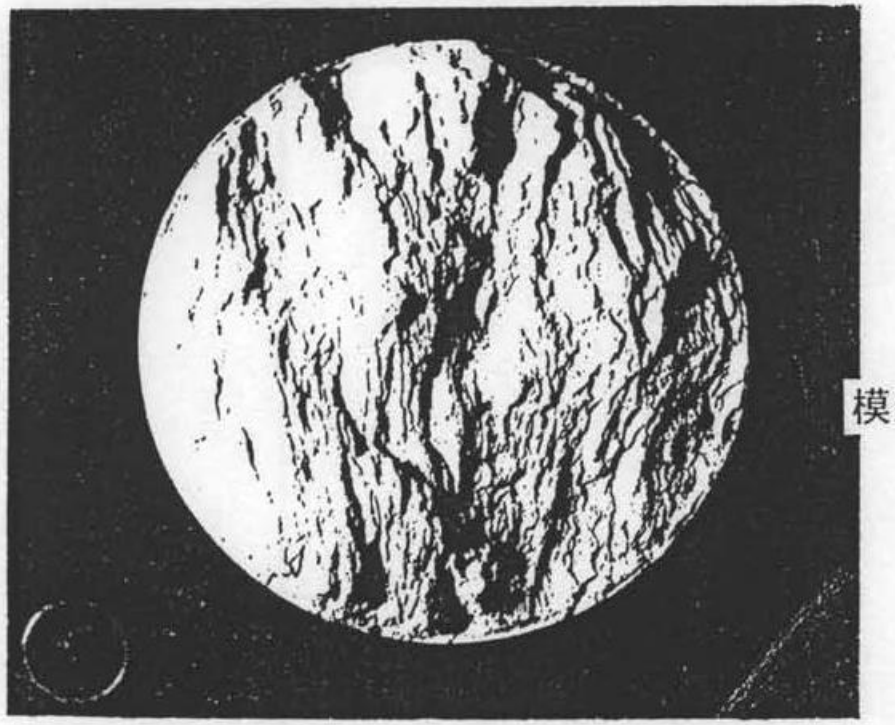


intersection lineation



Effect of shearing direction

不連續面強度之異向性



模型節理面試體

10cmφ 黑雲母片岩

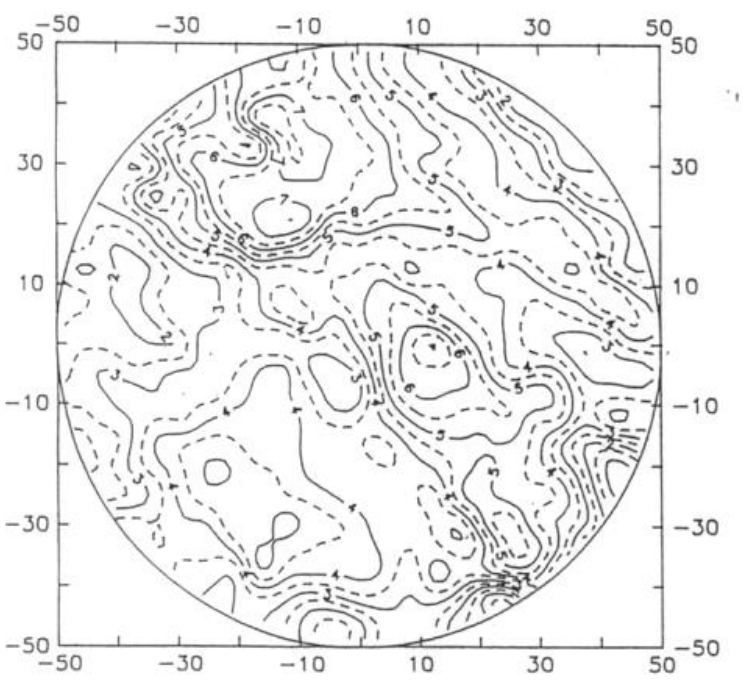
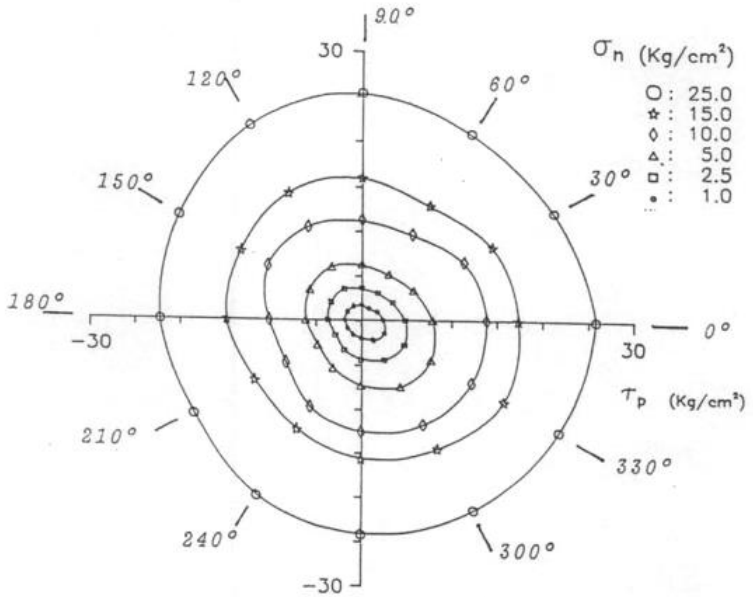


圖3.11 節理面等高線圖(a)正方形試體，(b)圓形試體(單位：mm)

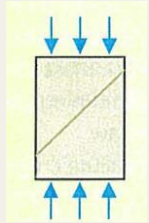


岩石節理面異向性(剪力強度)

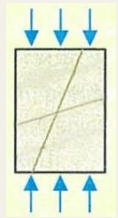
5.岩體的力學性質

1. 數學模式：Jaeger’s single plane of weakness theory, Bray’s superposition theory…
2. 室內實驗
3. 現地實驗
4. 數值模擬
5. 經驗統計法：**岩體分類法**

5.1 岩體之強度(數學模式)



+ 含單一條節理(裂面)之岩體強度
  a) intact rock fracture mode
  b) joint sliding mode
  c) Jaeger's single plane of weakness theory (1960)

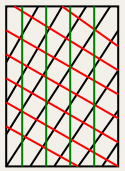


Vallejo and Ferrer (2011)

+ 含多條裂面之岩體強度
  a) Bray's superposition principle

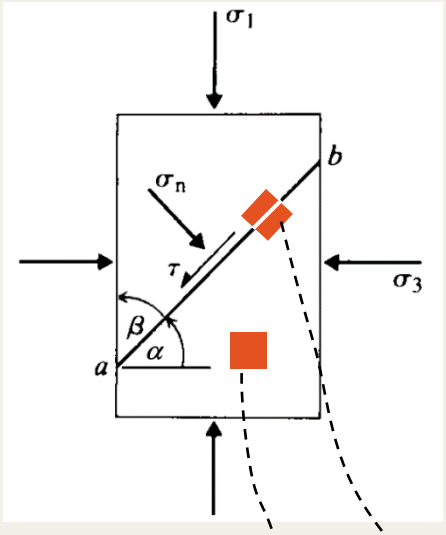
Hencher (2015)

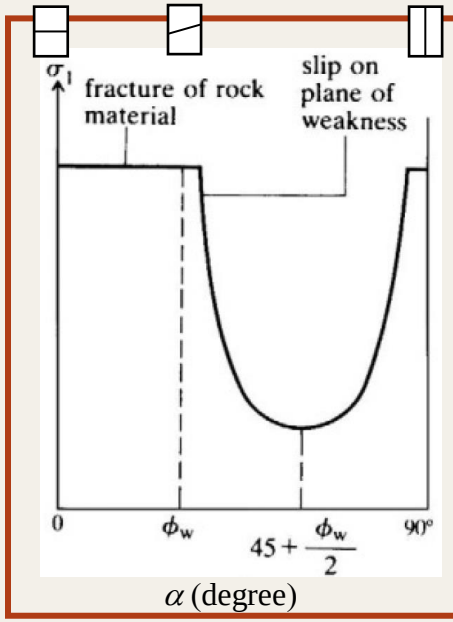
+ 含一組節理之岩體強度
  a) Intact rock fracture mode
  b) Joint sliding mode
  c) Mixed mode



+ 含多組節理之岩體強度
  a) 趨向isotropic ➔ Hoek-Brown failure criterion

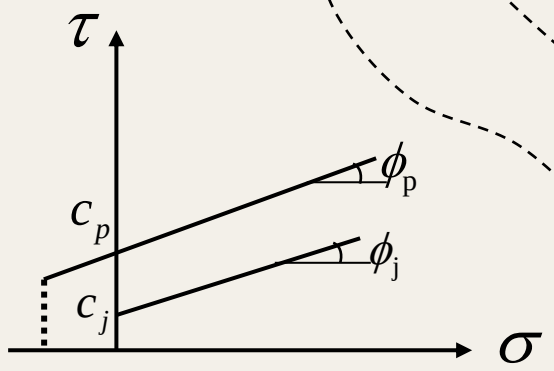
含單一裂面(弱面)之岩體強度





兩種破壞型態：

- 岩石材料破壞
  - 任一處剪應力>τ_f
- 沿著弱面滑動
  - 弱面上的剪應力>τ_j



- 岩石材料之破裂(c_p、ϕ_p)

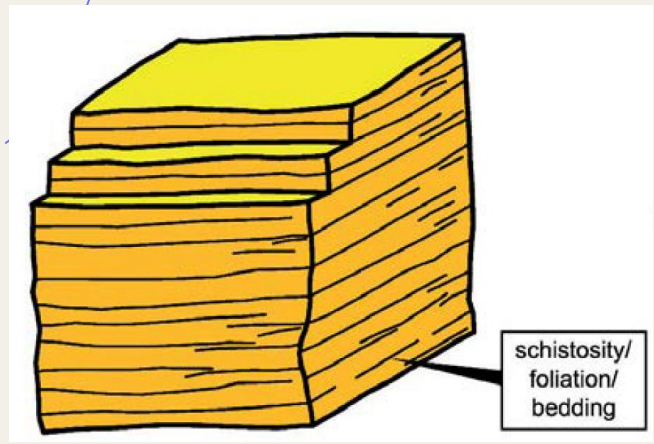
  $\tau_f = c_p + \sigma \tan\phi_p$

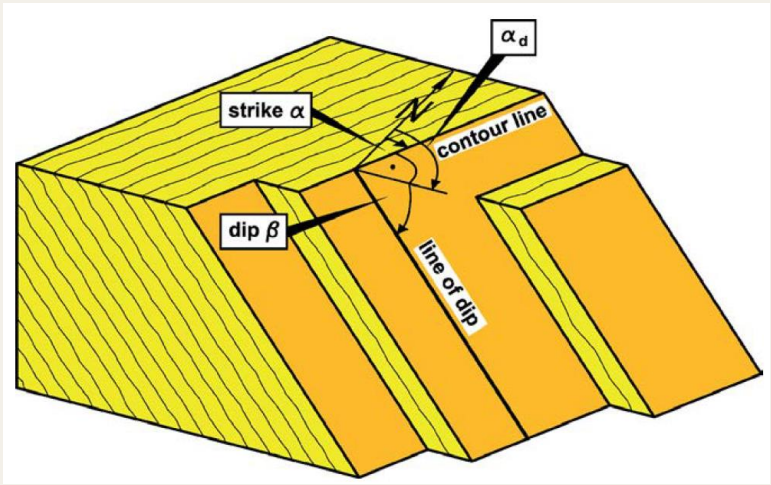
- 岩石弱面之滑動(c_j、ϕ_j)

  $\tau_j = c_j + \sigma \tan\phi_j$

含一組節理之岩體**強度**

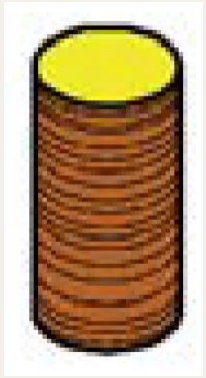
現場岩體





Wittke (2014)

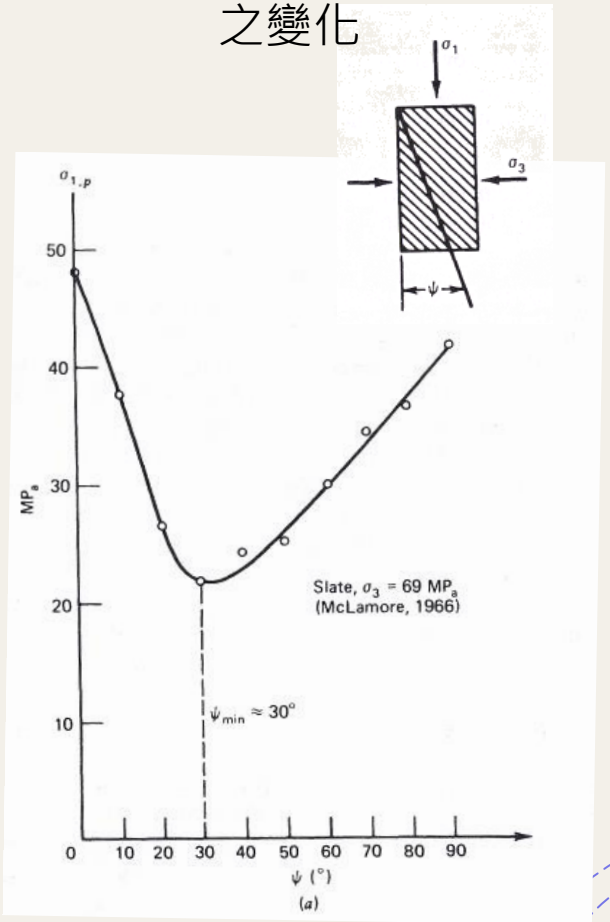
室內實驗



Wittke (2014)

Hencher (2015)

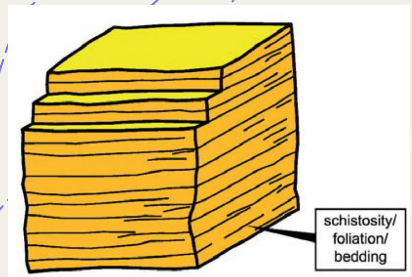
含一組節理岩體的**強度**隨不同ψ之變化



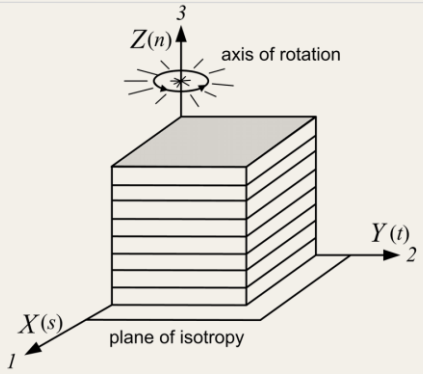
Goodman (1989)

岩體變形性

+岩體楊氏模數經驗公式之統整



Wittke (2014)



橫向等向性材料5個材料參數

E_n、G_{ns}、ν_{sn}、E_s、ν_{st}

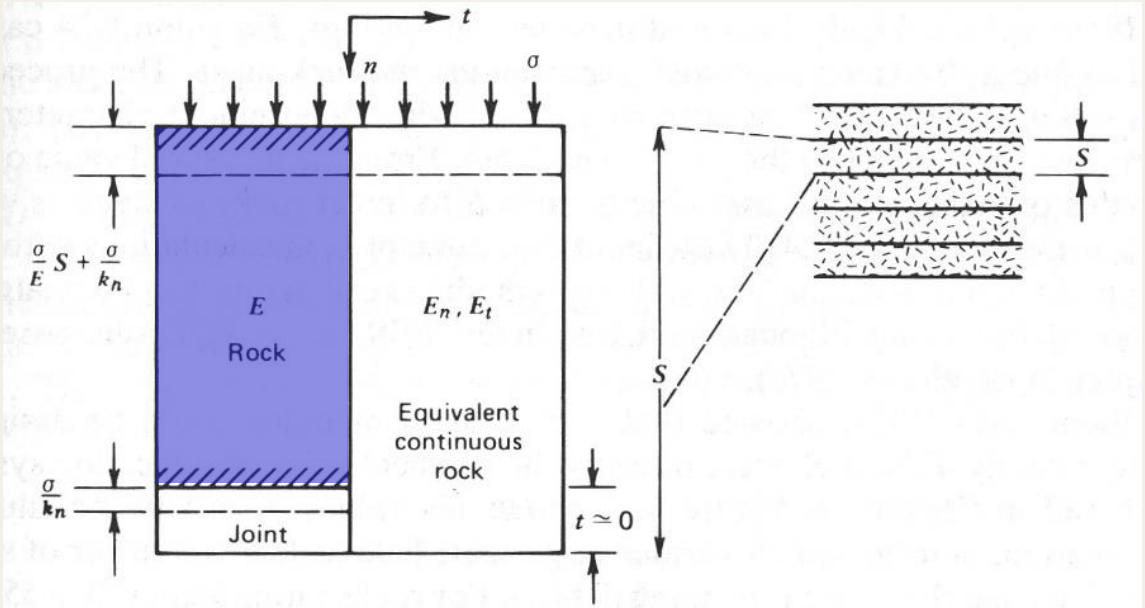


Table 3.25 EMPIRICAL EXPRESSIONS FOR ESTIMATING THE DEFORMATION MODULUS IN ISOTROPIC ROCK MASSES

Criterion	Application	
$E = 2\,RMR - 100$ (GPa) (Bieniawski, 1978)	— Good quality rock masses, RMR > 50–55 — Not valid for low quality rock masses	— Does not take laboratory data into account — E does not depend on the values of σ_{ci} or E_i
$E = 10^{(RMR-10)/40}$ (GPa) (Serafim & Pereira, 1983)	— Fair-low quality rock masses, 10 < RMR < 50 — Especially indicated for values 1 < E < 10 GPa — Too high values are obtained for low-very low quality rock masses	
$E = \left(1-\frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}}\,10^{(GSI-10)/40}$ (σ_{ci} in MPa; E in GPa) (Hoek *et al.*, 2002)	— Indicated for weak or soft rock masses with low or very low quality (GSI < 25) and intact rock with σ_{ci} < 100 MPa — D (disturance factor) = 0 (undisturbed rock mass) to 1 (fully disturbed)	— E is a function of the value of σ_{ci}
$E = 100{,}000\left(\frac{1-D/2}{1+e^{(75+25D-GSI)/11}}\right)$ (E in MPa) (Hoek & Diederichs, 2006)	— Valid for GSI = 20 to 80 — D = 0 (undisturbed rock mass) — D = 1 (fully disturbed)	— Properties of the intact rock are not considered — Recommended when reliable data for the intact rock are not available
$E = 10Q_c^{1/3}$ (E in GPa) (Barton, 1995, 2006)	— Indicated for jointed or fractured rock masses	— Intact properties of the rock are considered by means of σ_{ci}
$E = 10^{(Vp\,-\,0.5)/3}$ (V_p in km/s; E in GPa) (Barton, 2006)		— Rock mass properties are considered (V_p picks up density and strength)

E = Rock mass empirical static deformation modulus.
E_i = Intact rock deformation modulus measured in laboratory.
RMR = Rock Mass Rating.
GSI = Geological strength index.
Q = Quality index.
$Q_c = Q \cdot \sigma_c/100$ (σ_{ci} in MPa).
σ_{ci} = Intact rock uniaxial compressive strength.
V_p = P wave velocity.

- These correlations have not yet been sufficiently verified.
- They give approximate, merely indicative values.
- In general, they overestimate the value of the rock mass deformation modulus.
- They do not take the possible anisotropic nature of the *in situ* deformation modulus into account.
- For the rock mass, a range of values is recommended between 0.4E and 1.6E.

岩體變形與強度之量測與試驗

一. 靜態方法

1) 室內試驗
   + Uniaxial compressive test, Triaxial compressive test
2) 現地試驗
   a) Plate bearing test (平鈑載重試驗)
   b) Pressure tunnel test
      + Hydraulic pressure chamber test
      + Radial jacking test
   c) Borehole test
      + Borehole dilatometer
      + Borehole jack: Goodman's jack
   d) Flat jack test
   e) Back analysis

二. 動態方法

1) 室內試驗
   a) Resonance method
      + Longitudinal vibration
      + Flexural vibration
      + Torsional vibration
   b) Ultrasonic pulse method
2) 現地試驗

5.2 岩體室內試驗

+楊氏模數、單壓強度、巴西人張力強度之異向性

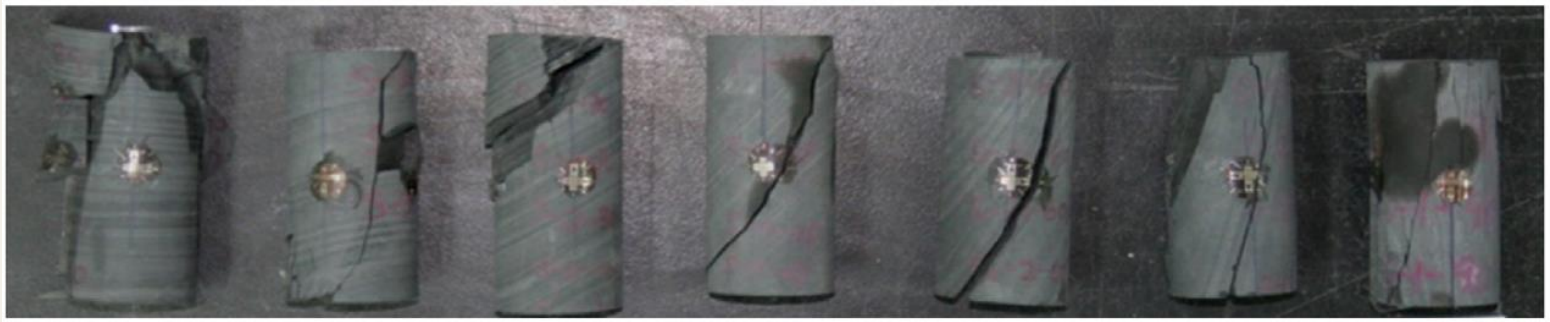


Fig. 9. Comparison of post failure specimens obtained from Boryeong shale laboratory tests (from [2], top) and bonded-particle DEM modeling (bottom). After (a) uniaxial compressive strength tests and (b) Brazilian tensile strength tests. (For interpretation of references to color in this figure, the reader is referred to the web version of this article.)

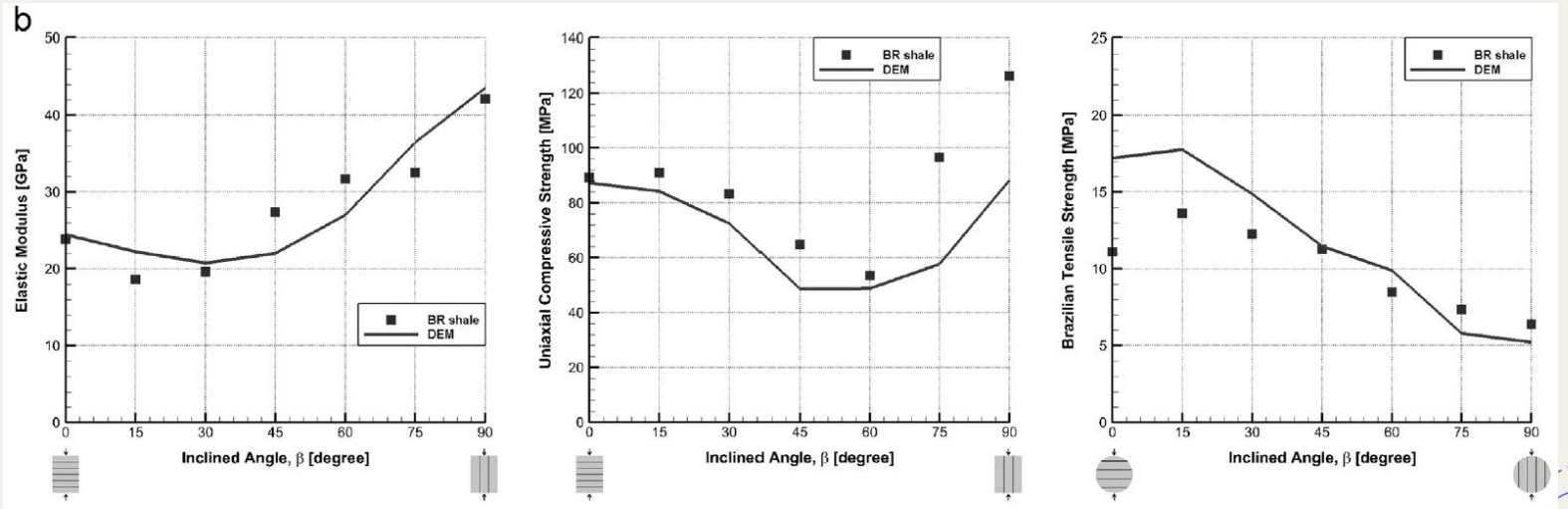


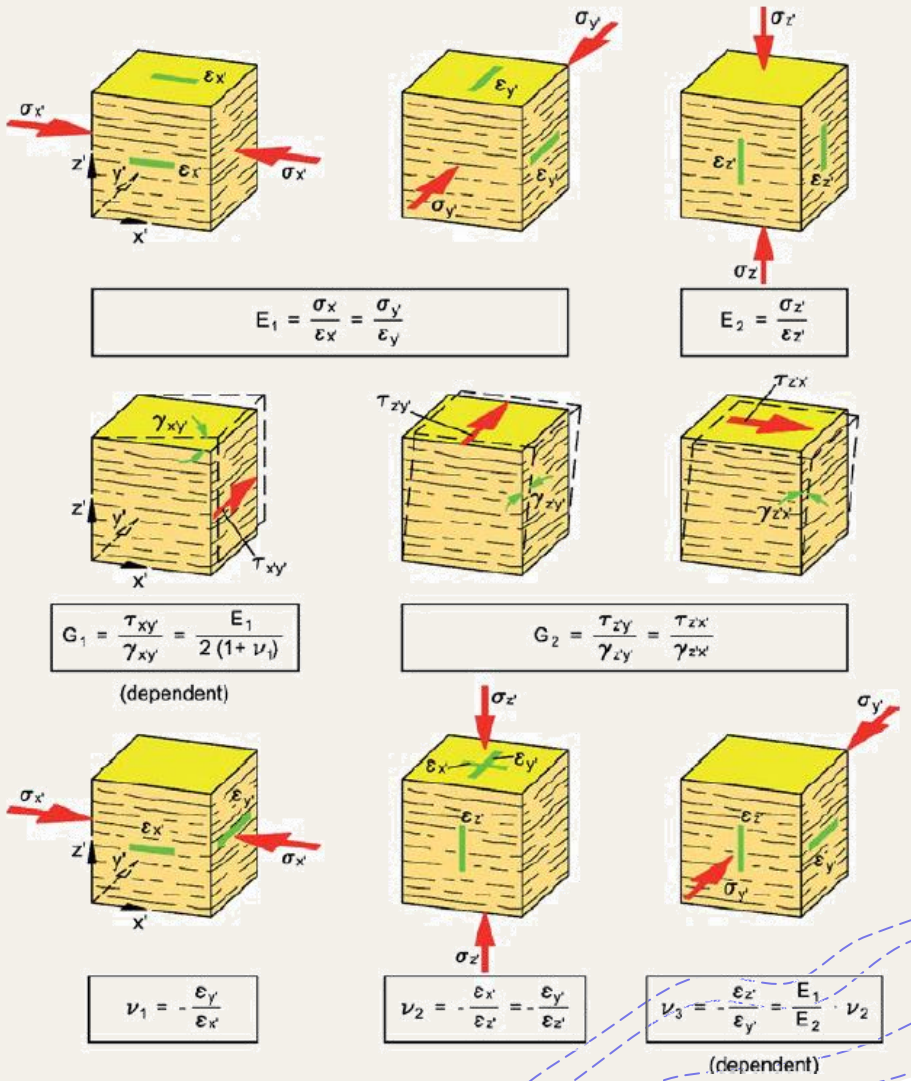
Fig. 7. Comparison of elastic modulus, uniaxial compressive strength and Brazilian tensile strength from laboratory tests (symbol), and those from bonded-particle DEM modeling (line). (a) Asan gneiss, (b) Boryeong shale, and (c) Yeoncheon schist.

保寧頁岩 Boryeong shale (南韓)

Park and Min (2015)

5.2 岩體室內試驗

- 橫向等向性材料
- 左圖中有兩個參數相關(dependent)，獨立變數共5個 E_n、G_{ns}、ν_{sn}、E_s、ν_{st}



5.3岩體現地試驗

+Plate bearing test平鈑載重試驗

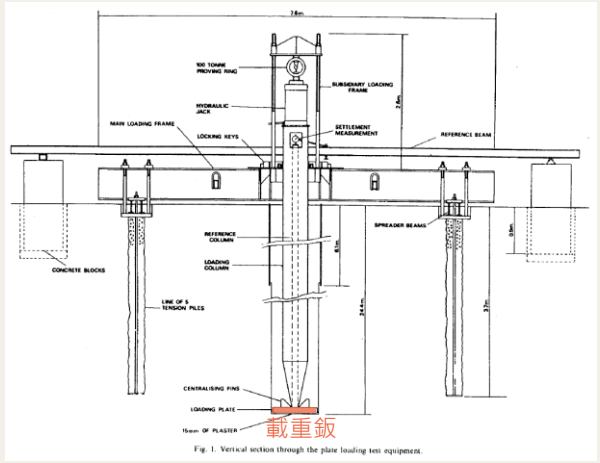


Fig. 1. Vertical section through the plate loading test equipment.

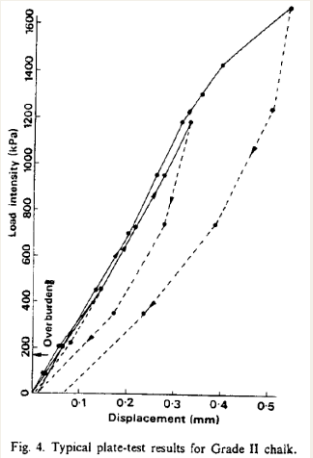
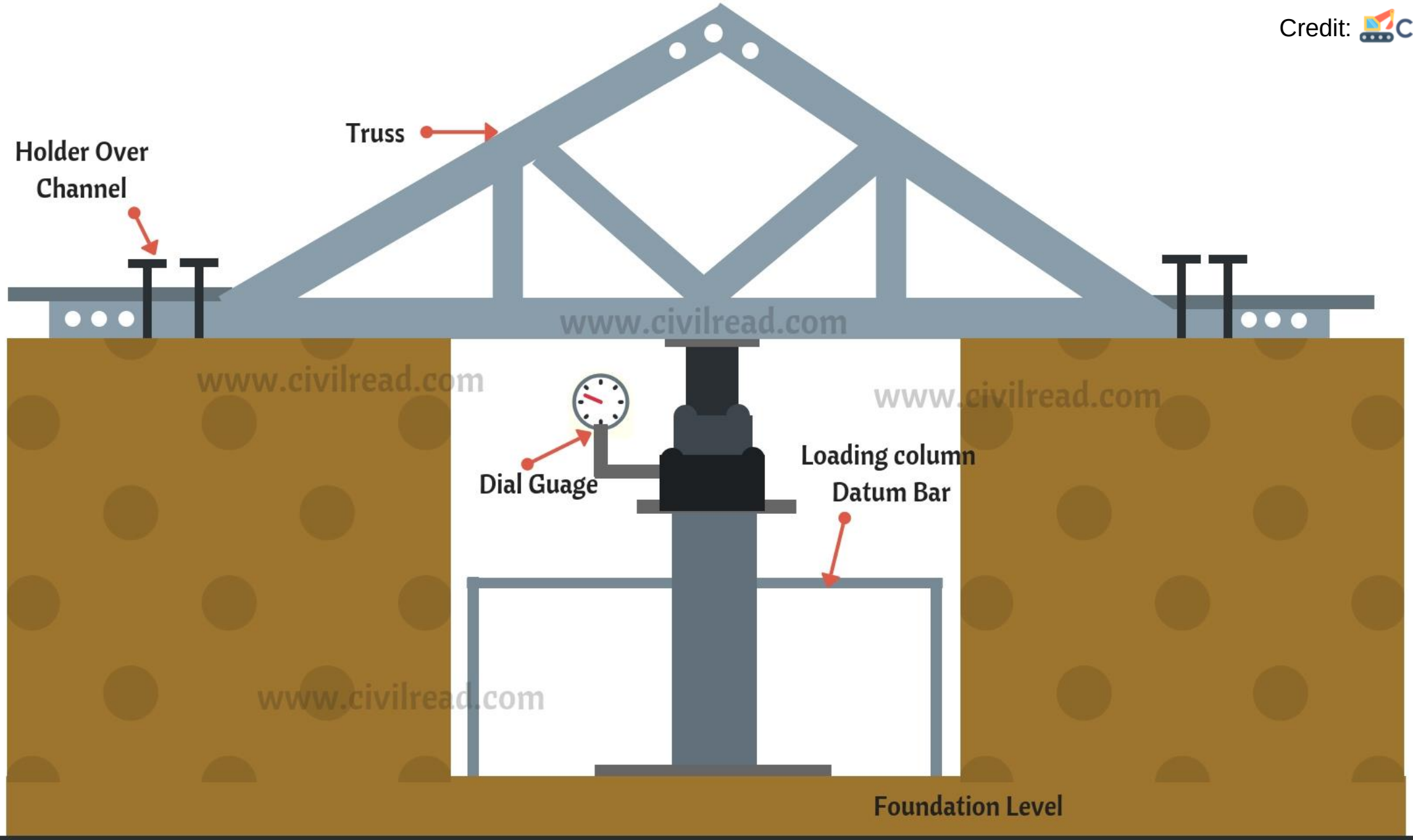


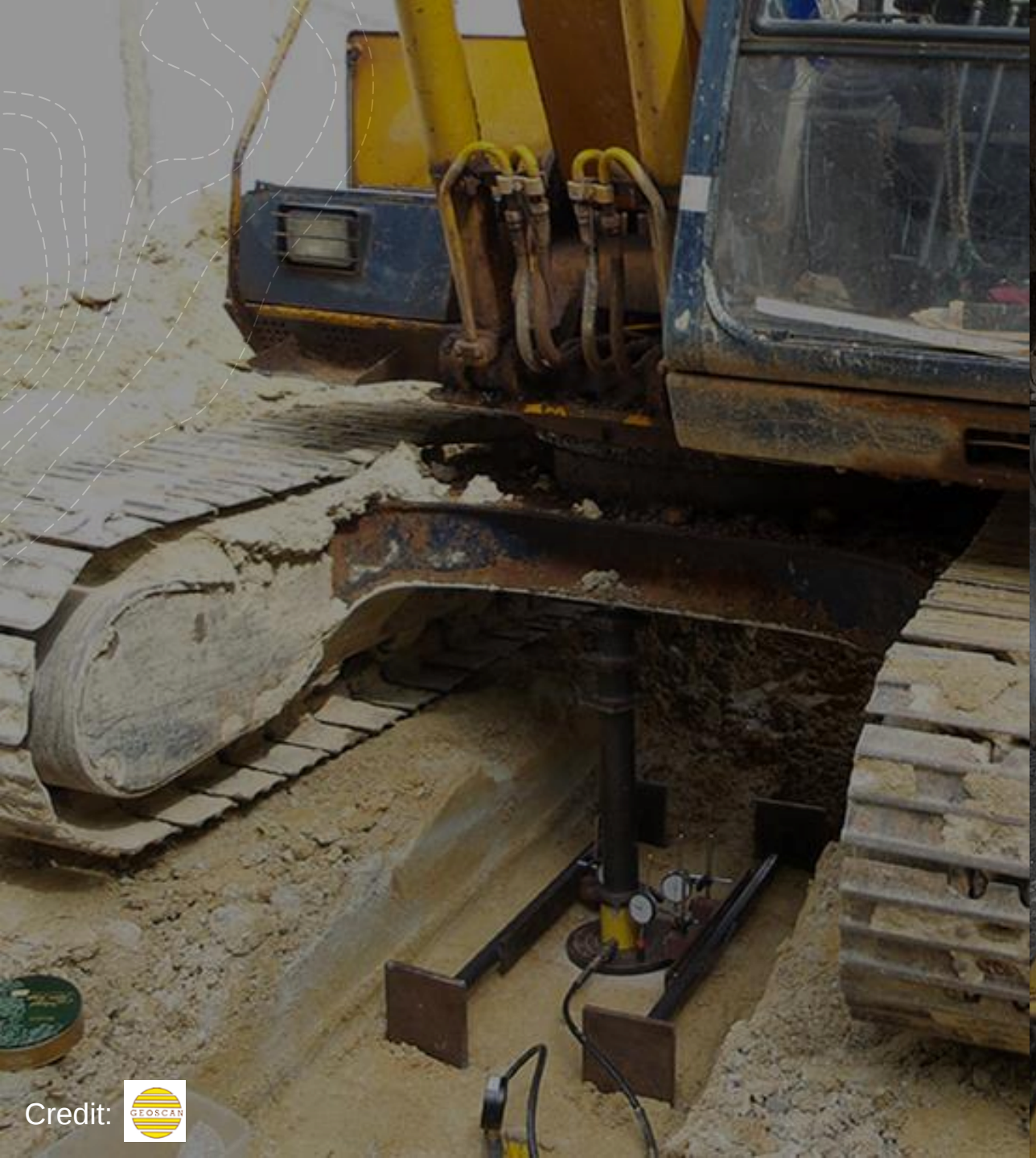
Fig. 4. Typical plate-test results for Grade II chalk.

ISRM (1981)

Credit: CivilRead



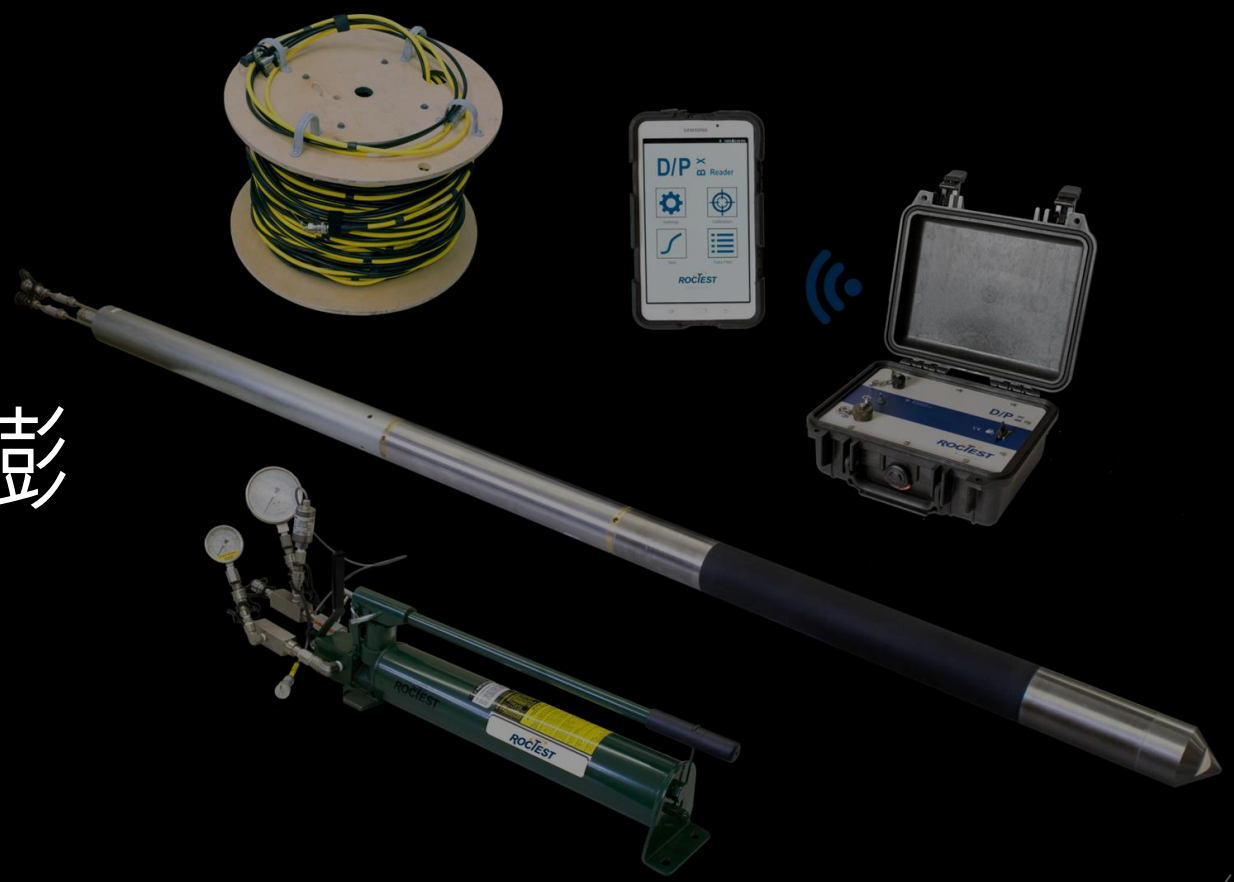
Reaction Truss Method - Plate Load Test



Credit: GEOSCAN

Credit: SUB SURFACE Site Investigation Specialists, Geotechnical & Environmental Consultants

Borehole Dilatometer 孔內膨脹試驗



Credit: ROCTEST

節理間距非常大

Calculation of deformability parameters of rock

12. (a) For any segment of the pressure displacement diagram (e.g. Fig. 7) and in rock with widely spaced joints [see 12(b) below], the corresponding secant dilatometric modulus E_d may be calculated as follows:

where:
$$E_d = (1 + \nu_R) D \frac{\Delta p_i}{\Delta D} \text{ (MPa)}$$

Δp_i = pressure increment within the considered segment (MPa);
ΔD = corresponding average change in drillhole diameter D (m);
ν_R = Poisson's ratio of the rock mass.

節理間距很密，岩體破碎

(b) If the test is performed in cracked rock, and if p_i exceeds about twice the average ground pressure p_0 around the drillhole, all existing radial cracks will open, and the equation of paragraph 12(a) is to be replaced by:

$$E_d = D \frac{p_i}{\Delta D} (1 + \nu_R) \left[(1 - \nu_R) \ln \left(\frac{p_i}{2p_0} \right) + 1 \right] \text{ (MPa)}$$

where:

p_i = applied pressure (MPa), and
ΔD = average increase of drillhole diameter (m), when pressure increases from zero to p_i.

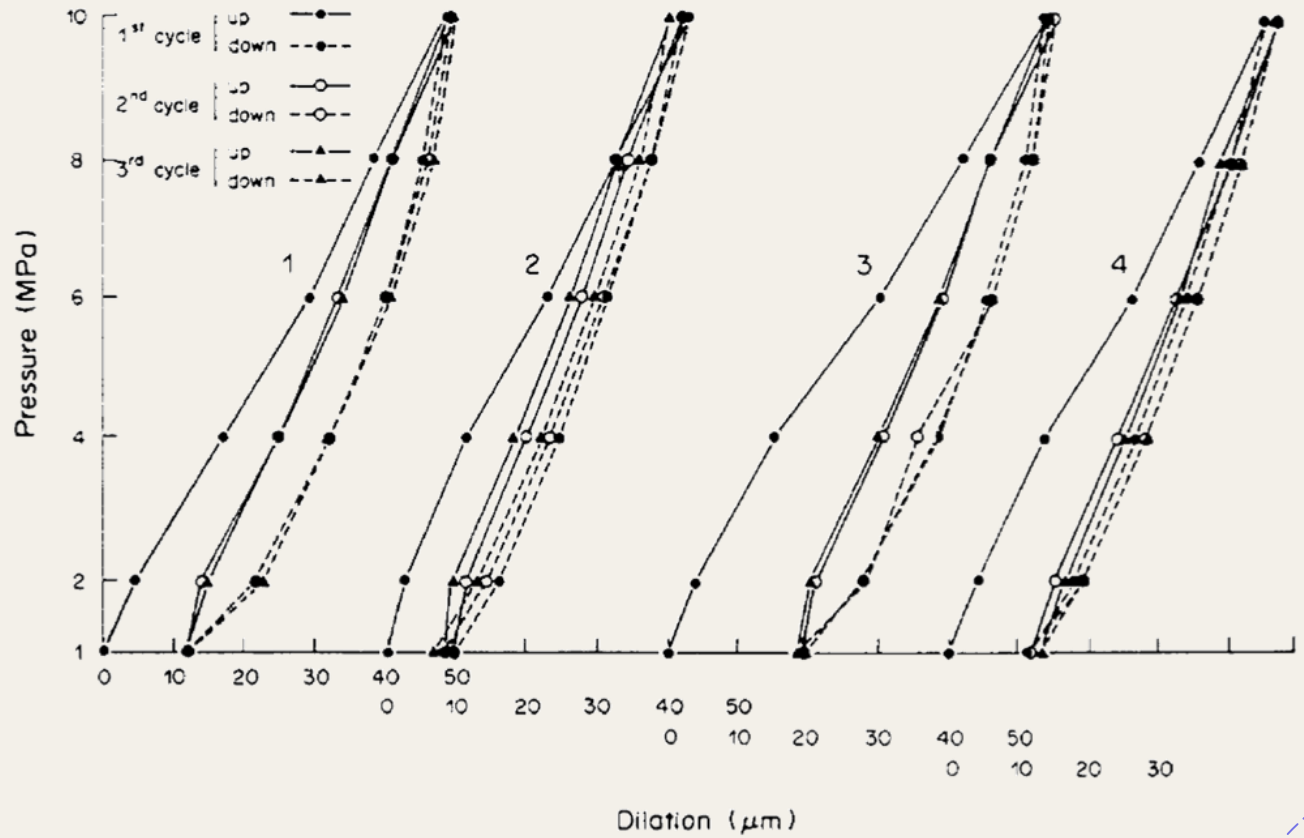


Fig. 7. Pressure–dilation curves, showing separate curves for each of four dilation-measuring transducers mounted in the same probe (LNEC dilatometer, [2, 20]).

5.4岩體的數值模擬

FEA(Finite Element Analysis)

+有限元素法FEM(Finite Element Method), 如Abaqus, Plaxis, RS2, RS3…

+有限差分法FDM(Finite Difference Method), 如FLAC

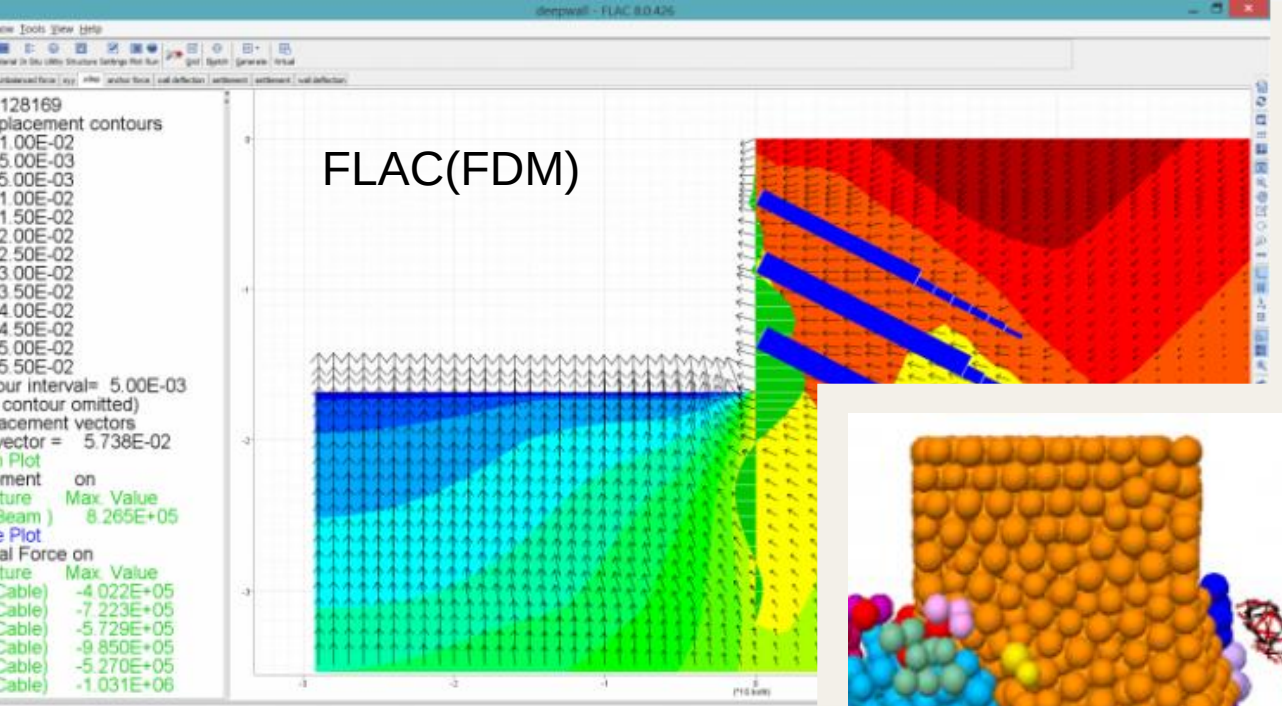
+邊界元素法BEM(Boundary Element Method)

+離散元素法DEM(Distinct Element Method), 如UDEC, PFC2D, PFC3D…

+不連續變形分析DDA(Discontinuous Deformation Analysis)

+無網格法Mesh-free method

+混合(以上兩種或以上之分析)法Hybrid method


128169
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our interval= 5.00E-03
contour omitted)
acement vectors
ector = 5.738E-02
Plot
ment on
ture Max. Value
Beam) 8.265E+05
Plot
al Force on
ture Max. Value
Cable) -4.022E+05
Cable) -7.223E+05
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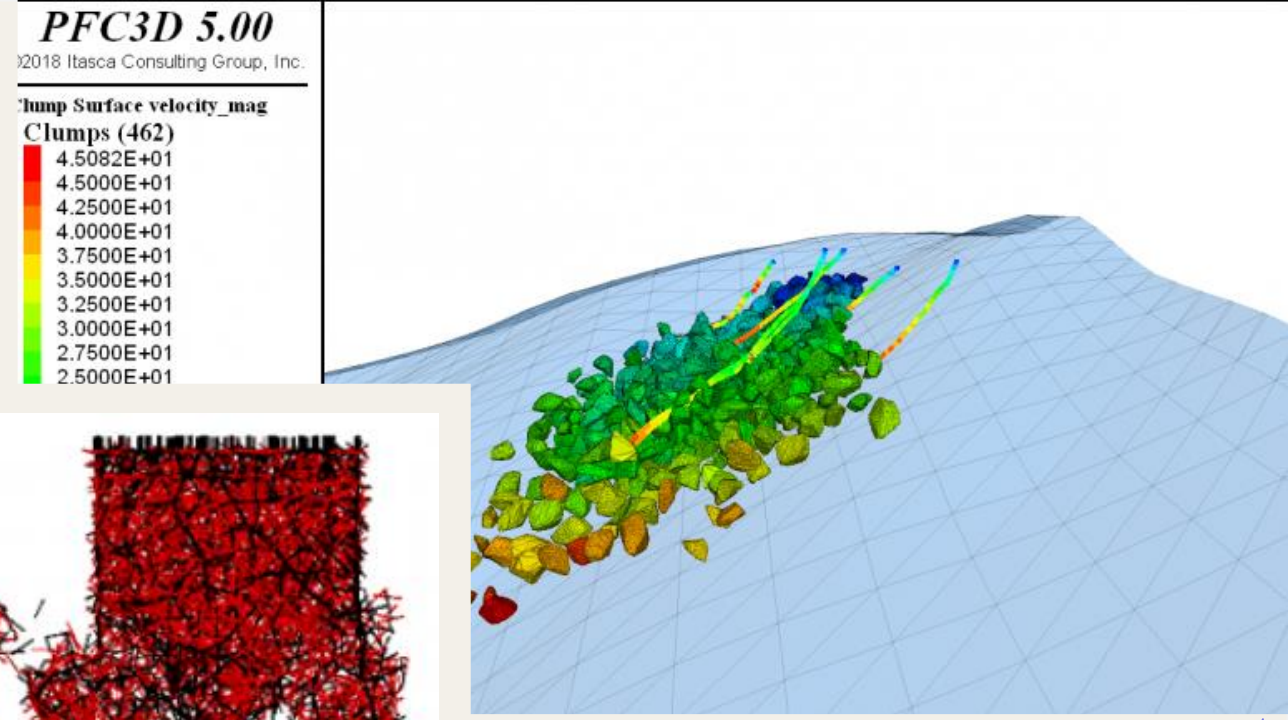
Cable) -9.850E+05
Cable) -5.270E+05
Cable) -1.031E+06
FLAC(FDM)


PFC3D 5.00
2018 Itasca Consulting Group, Inc.
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Clumps (462)
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4.5000E+01
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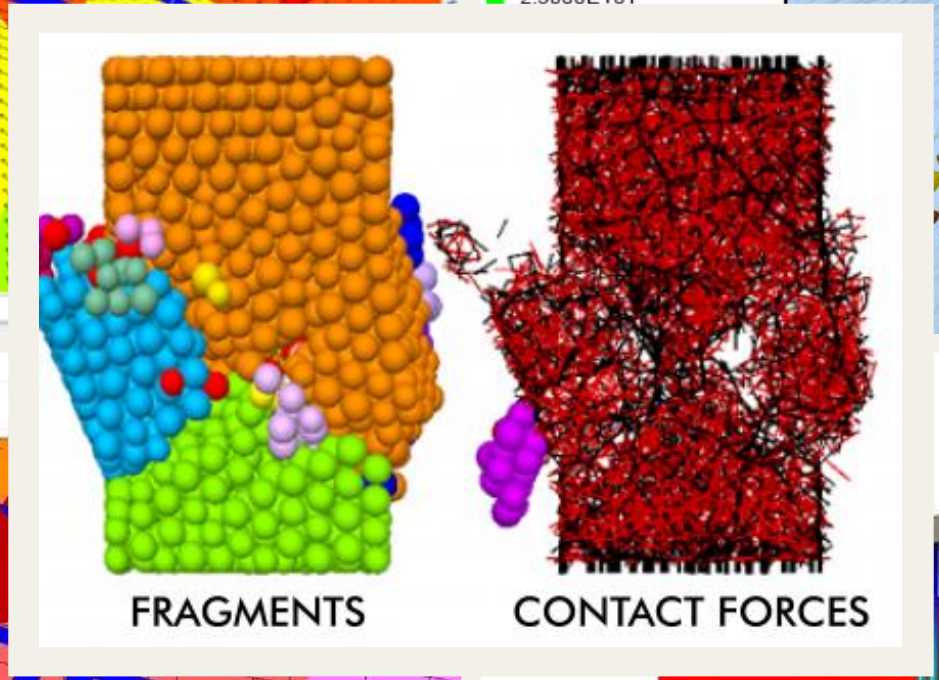
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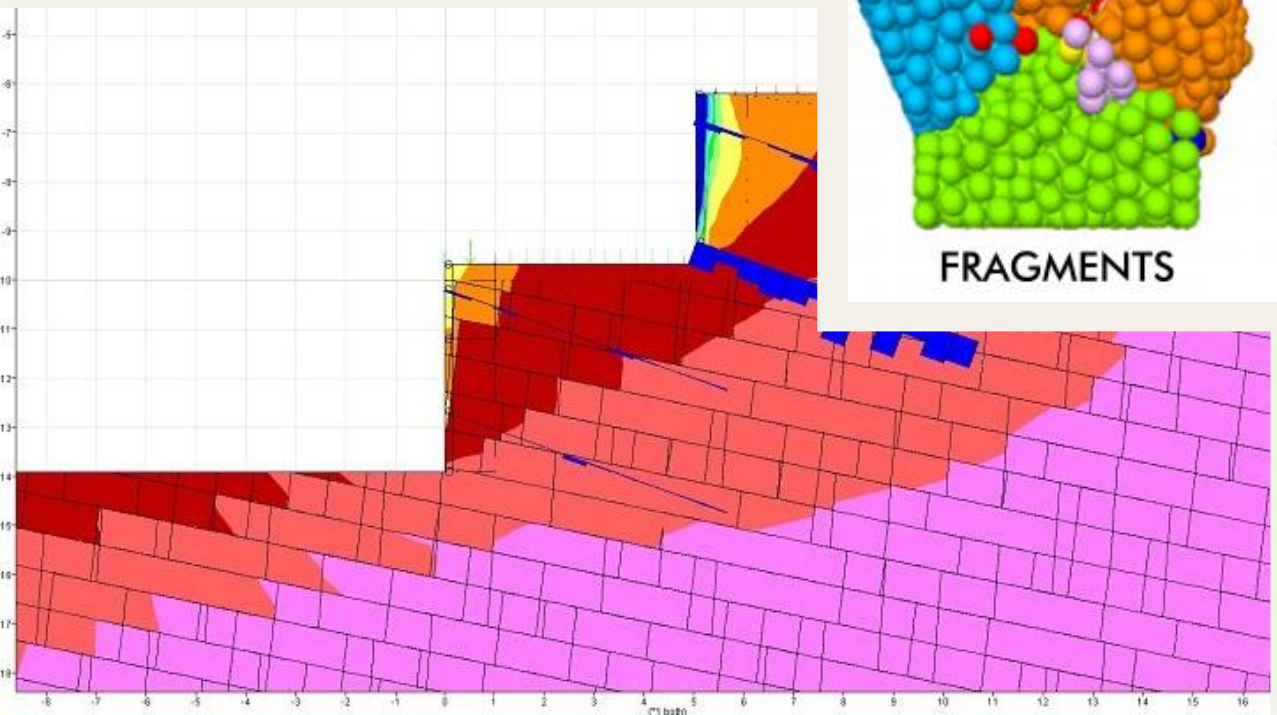
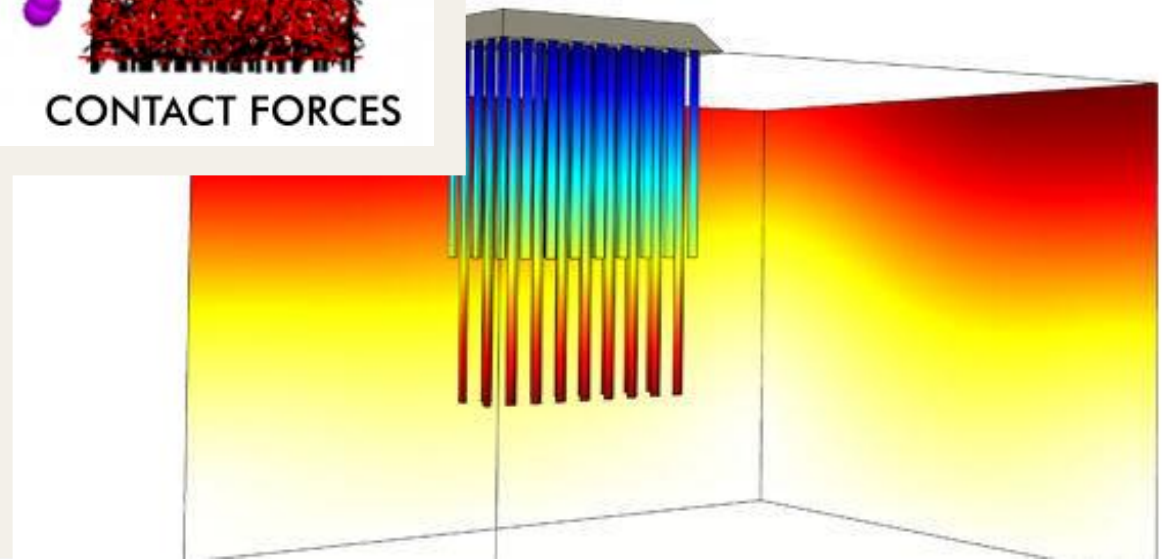
CONTACT FORCES





5.5岩體分類法 參考**第二章**

- 我國常見之岩體分類法包含：
  - 南非CSIR的Bieniawski之岩體評分法 (Rock Mass Rating, RMR)
  - 挪威NGI的Barton等人之Q法 (Q-system)
  - 台灣岩體分類法(台灣地區岩體分類系統, PCCR)
  - 坡體評分法 (Slope Mass Rating, SMR)
  - 其他，如美國Wickham等人提出之岩石構造評分法(Rock Structure Rating, RSR)