National Technical University Athens (NTUA)
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Field Measurements for Assessing the Stability of Tunnels and Slopes

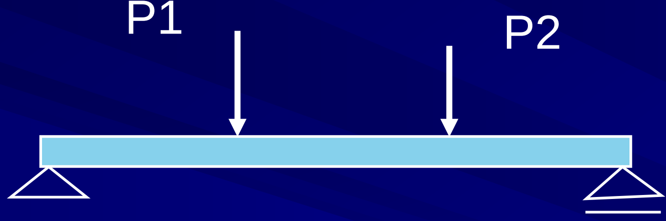
S. Sakurai
President, CERIF
Past President of ISRM
Prof. Emeritus, Kobe University &
Hiroshima Institute of Technology

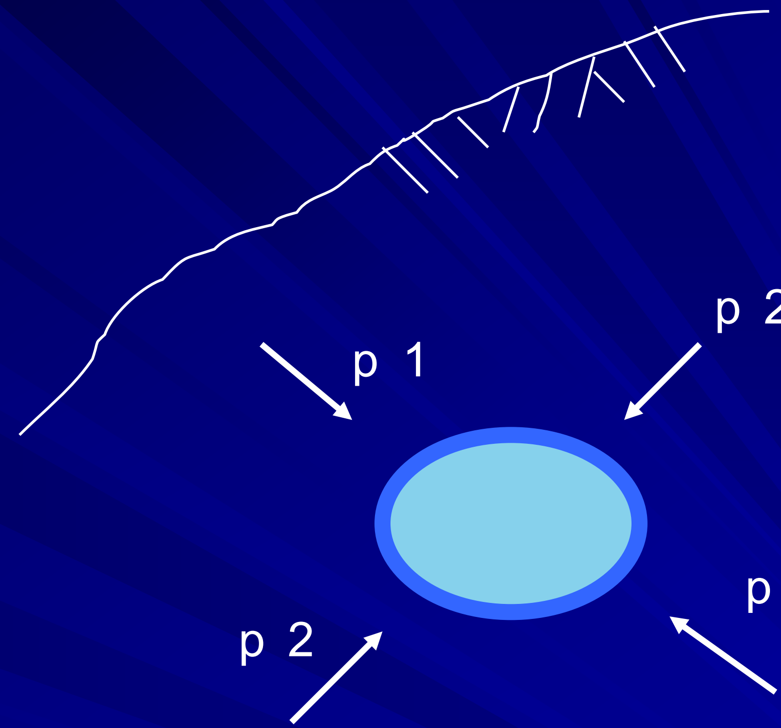
Contents

Design Concept of bridge and Tunnel

(1) Bridge

(2)Tunnel





$$\sigma = f(P_1, P_2, E)$$

$$\sigma \le \sigma_a$$

σ_a : **Allowable Stress**

$$\sigma = g(p1, p2, E, \nu, c, \phi \qquad)$$

$$\varepsilon = h(p1, p2, E, \nu, c, \phi$$

$$\sigma \le \sigma_a$$

$$\boxed{\varepsilon \le \varepsilon_a}$$

(3) Design concept of slope

- Factor of safety:

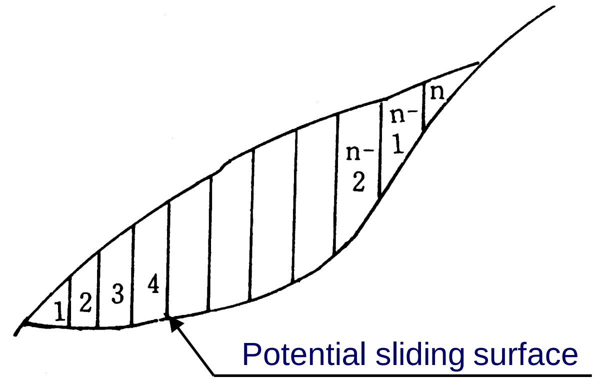
$$F = \frac{\Sigma R_i}{\Sigma T_i} \geq 1.0$$

where ΣR_i : shear strength

ΣT_i : sliding force

$$R_i = f(c, \phi \ldots\ldots)$$

Limit Equilibrium Analysis



$T_i = W_i \cdot \sin\theta_i$

$R_i = (W_i \cdot \cos\theta_i - U_i)\tan\phi + cl_i$

W_i：分割片の重量（t／m）

θ_i：すべり面の分割片部における傾斜角(度)

U_i：分割片に働く間隙水圧（t／m）

l_i：分割片のすべり面延長（m）

- We must estimate strength parameters (c, φ) .

Monitoring Approach

- For tunnels;

  Strain-based monitoring

- For slopes;

  Stress-based monitoring

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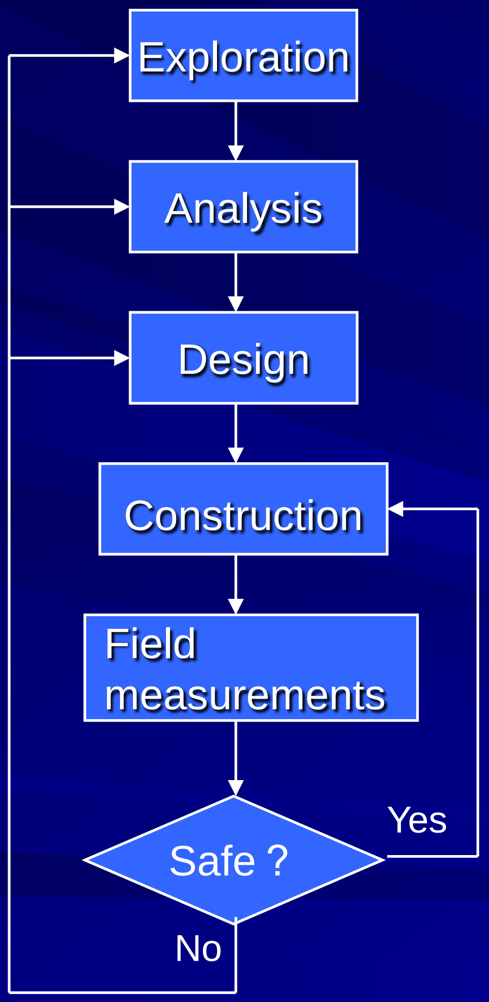
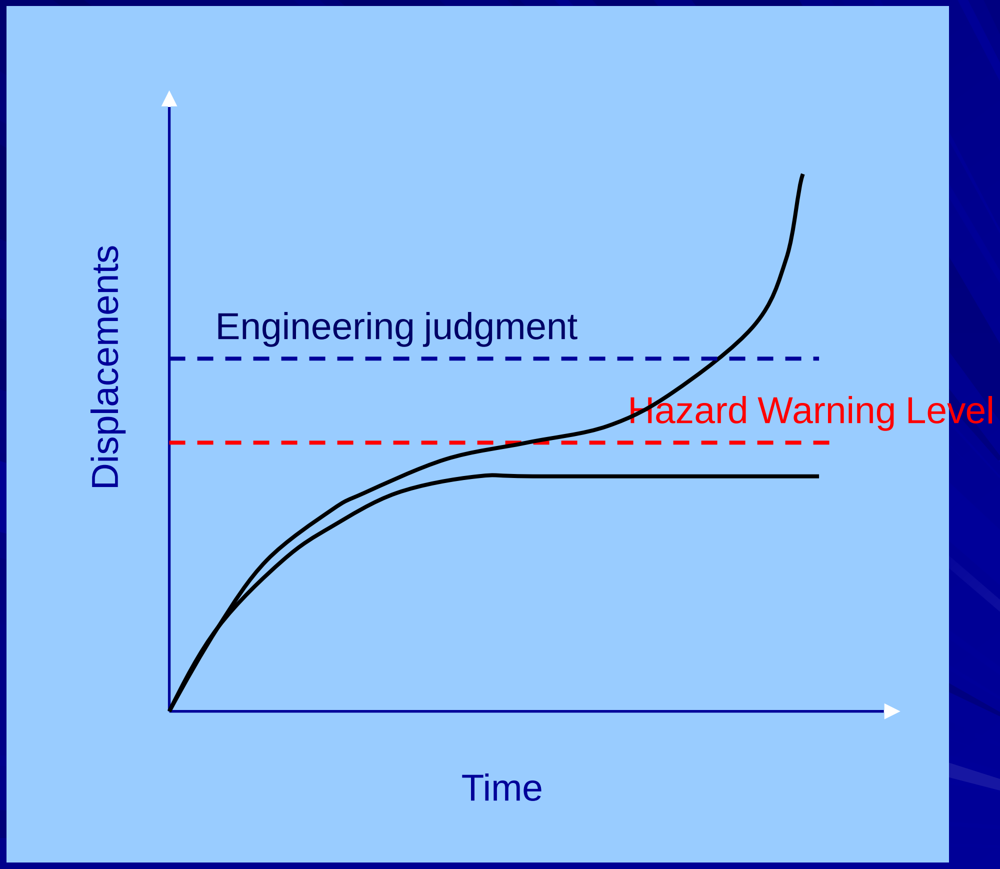
- Back analysis
- Anisotropic parameter
- Observational method in slopes
- Case study (Tunnel)
- Case study (Slope)
- Conclusions

Observational method





Strain can be easily determined from the measured displacements.

$$\varepsilon = \frac{u}{}$$

Where u : measured displacements

x: coordinate of measuri

We do not need any mechanical properties and initial stress of the ground

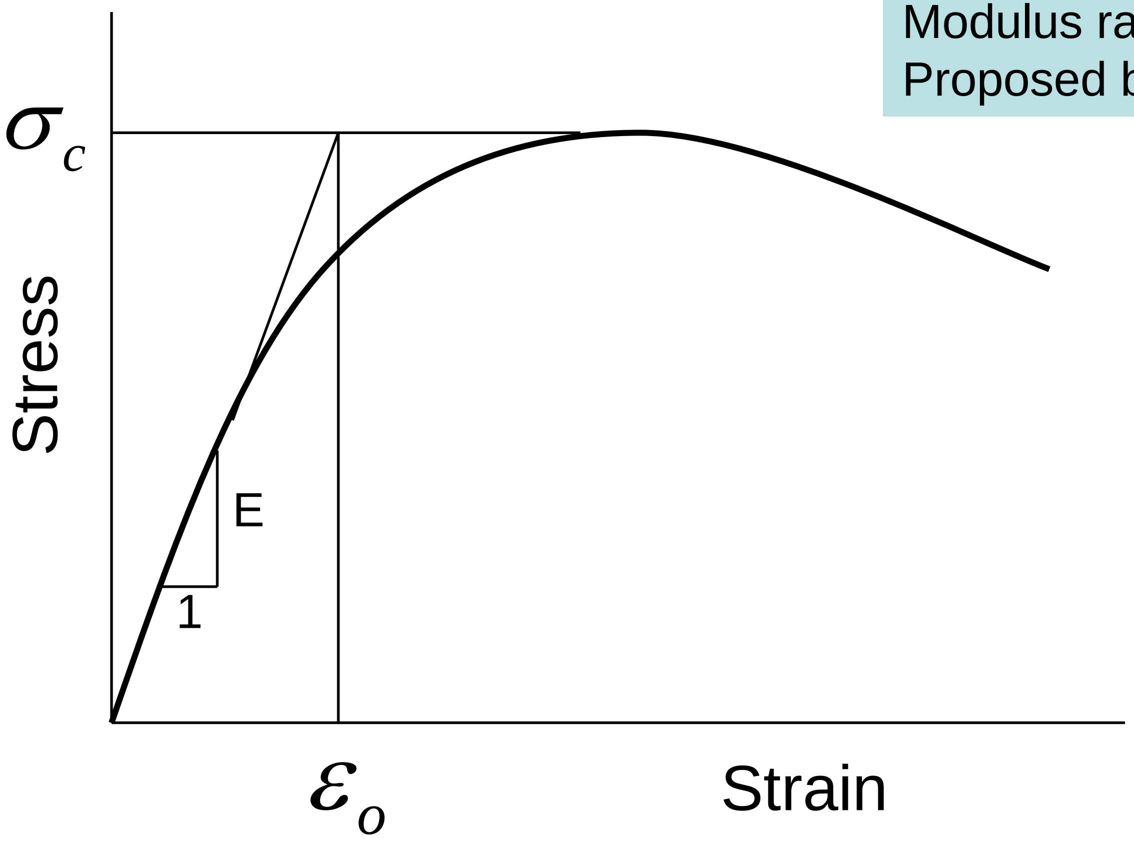
$$\varepsilon \leq \varepsilon_{\mathbf{0}}$$

where $\varepsilon_{\mathbf{0}}$: Critical strain (Allowable strain)

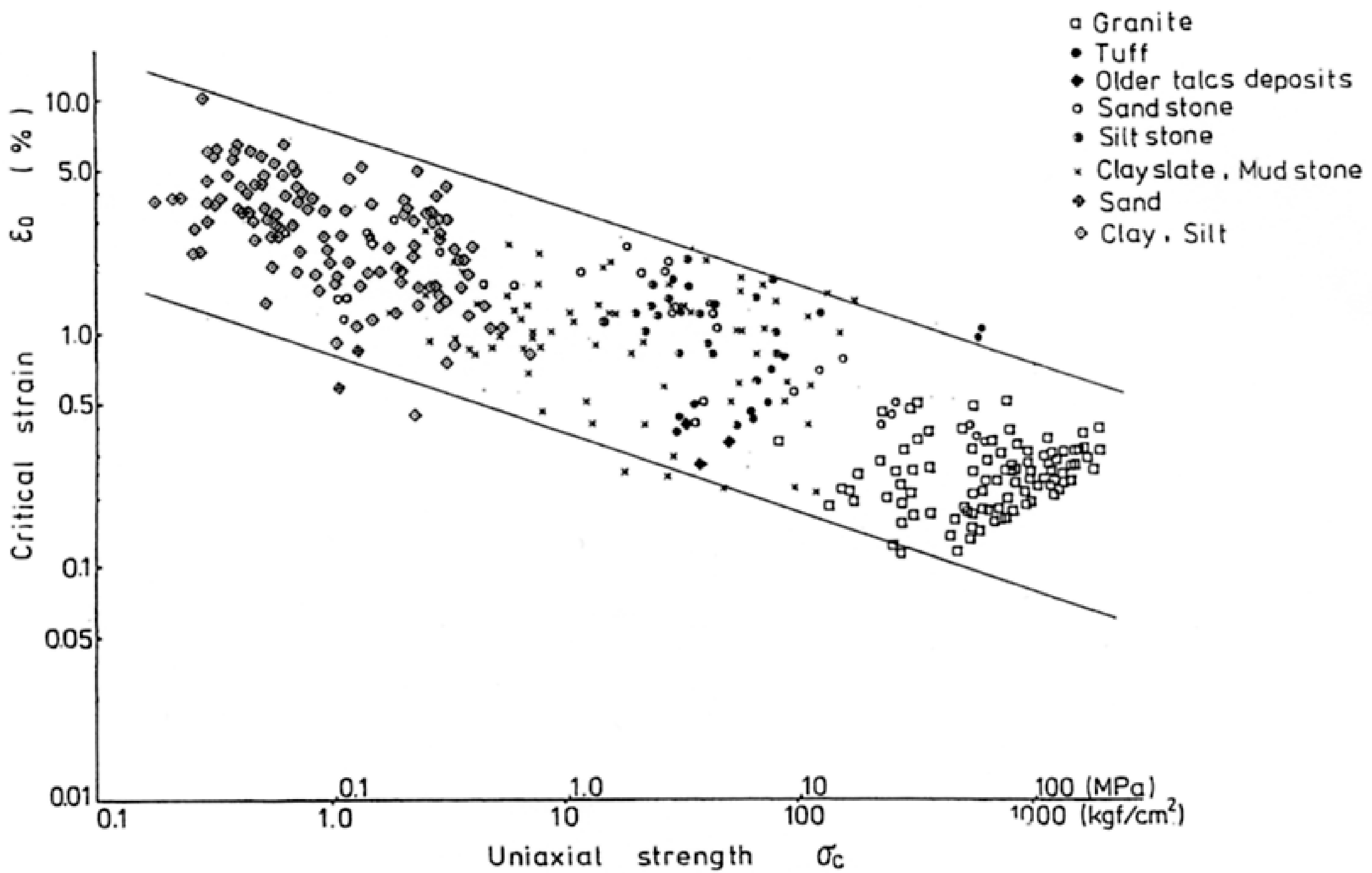
Definition of critical strain

Critical strain $\varepsilon_0 = \sigma_c / E$

Modulus ratio = E / σ_c
Proposed by Prof. Deer

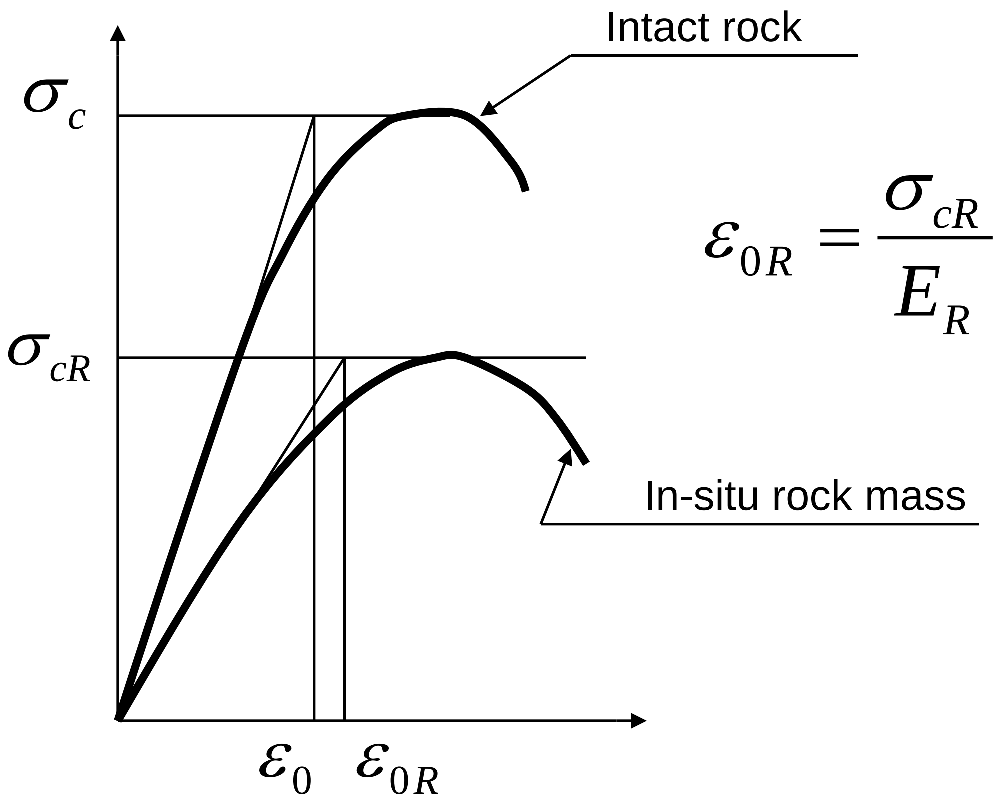


Critical strain obtained by laboratory tests



Relationship between critical strain and uniaxial strength of soils and rocks

Relationship between Critical Strains of Intact Rock and In-situ Rock Mass

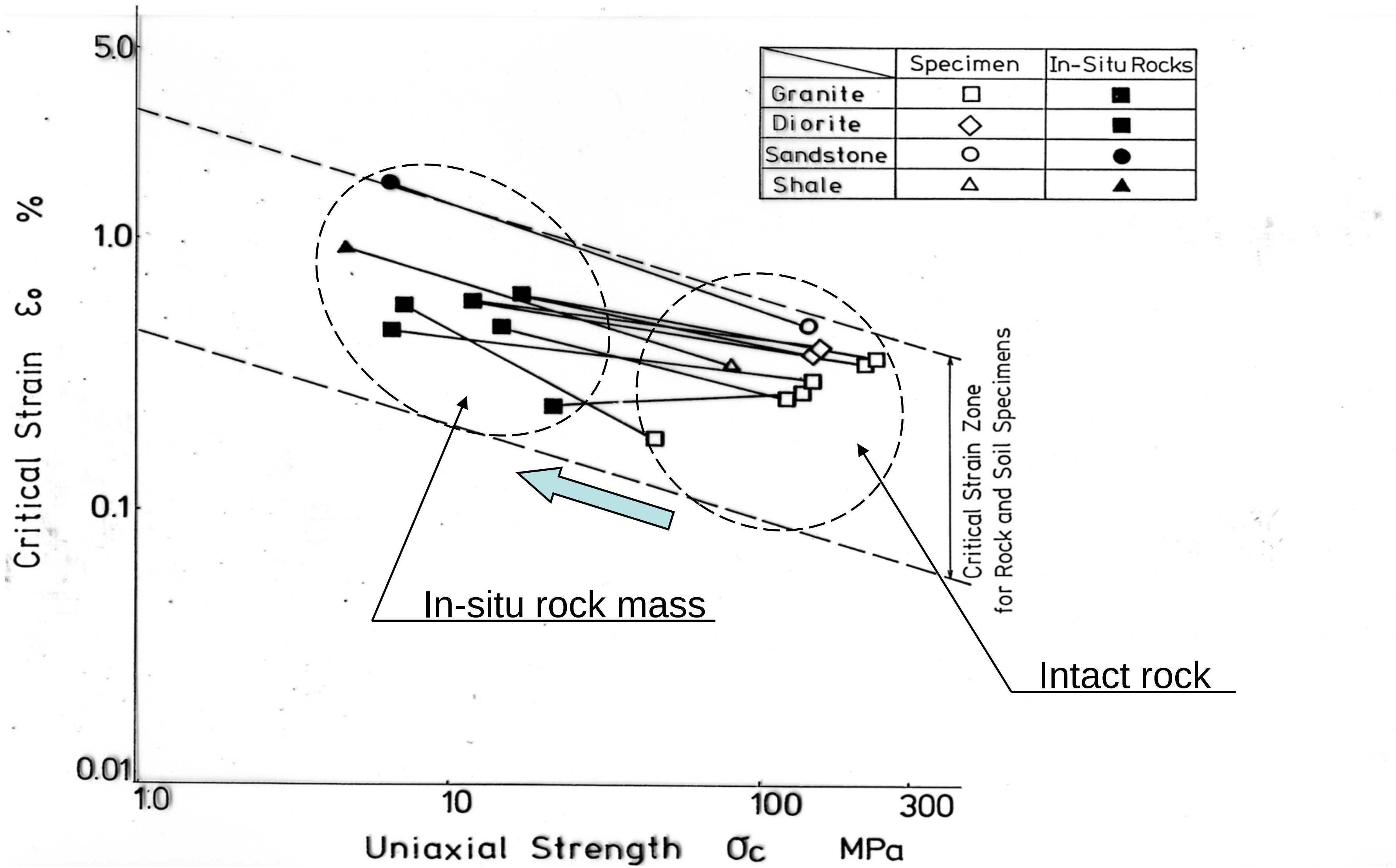


$$\varepsilon_{0R} = \frac{\sigma_{cR}}{E_R} = \frac{m\sigma_c}{nE_c} = \left(\frac{m}{n}\right)\varepsilon_0$$

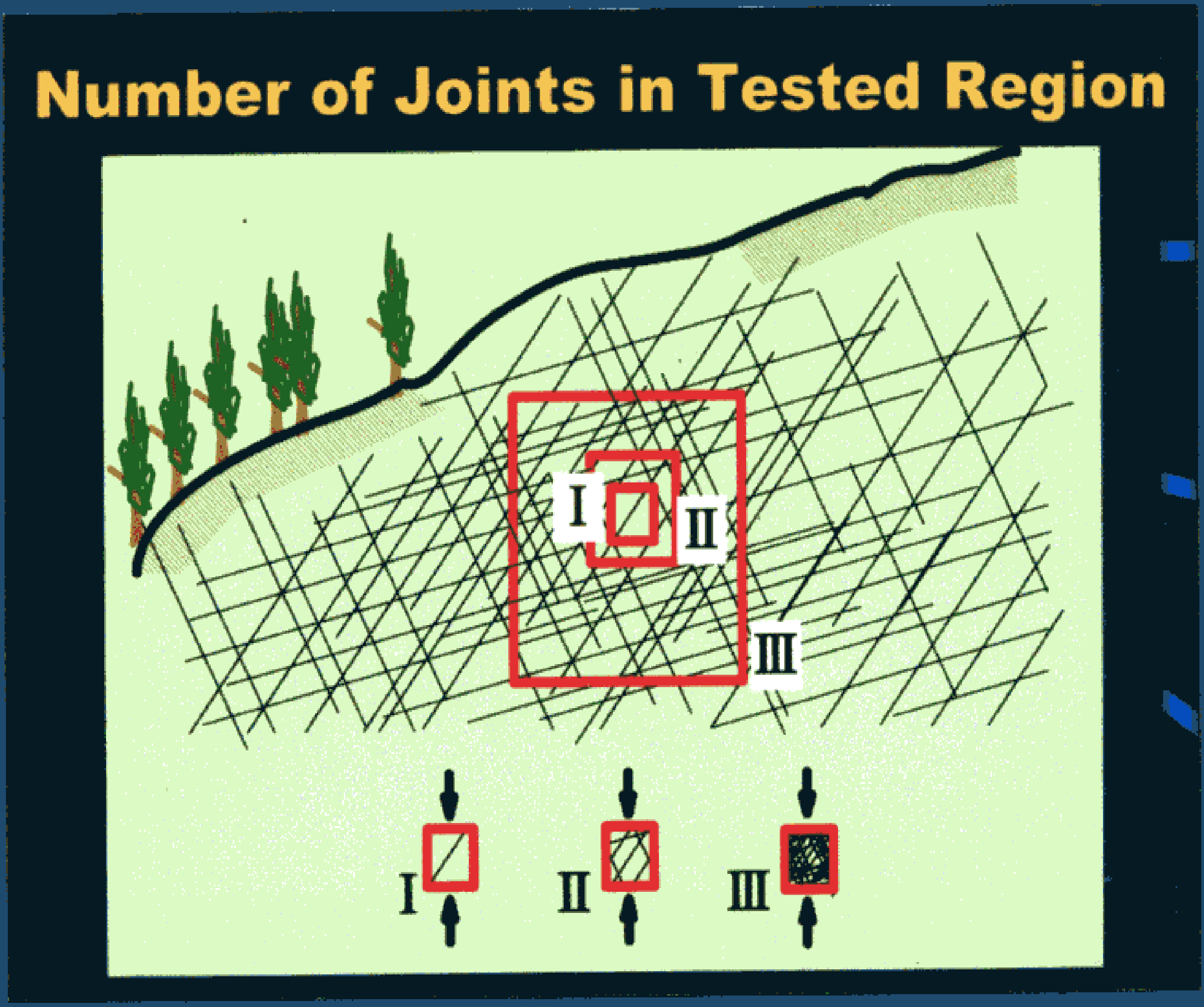
Strength and Deformability of Rocks and Rock Masses

Rock Type & Class	Rock Core			Rock Mass			Core vs Mass		
	σ_c MPa	E_c GPa	ε_0 %	σ_{cR} MPa	E_m GPa	ε_{0R} %	m	n	m/n
granite CH	227.4	62.7	0.362	16.70	2.65	0.631	0.0734	0.0422	1.74
granite CM	211.5	59.8	0.534	11.60	1.96	0.592	0.0548	0.0328	1.67
granite CL	141.8	47.1	0.301	6.39	1.37	0.466	0.0451	0.0292	1.54
diorite CH	145.2	37.3	0.389	16.70	2.65	0.631	0.1150	0.0711	1.62
diorite CM	153.5	38.2	0.402	11.60	1.96	0.592	0.0755	0.0513	1.47
granite B	130.2	47.5	0.274	20.97	8.60	0.244	0.161	0.181	0.89
granite CH	117.6	44.3	0.265	14.31	2.98	0.480	0.198	0.067	1.81
granite CM	44.7	23.7	0.189	6.99	1.21	0.578	0.156	0.051	3.06
sanastone ---	137.2	28.4	0.483	6.27	0.381	1.642	0.046	0.0134	3.43
shale ----	78.4	21.6	0.364	3.82	0.411	0.930	0.049	0.0191	2.57
sandstone	137.8	54.1	0.255	7.89	1.96	0.403	0.057	0.0362	1.57
sandstone	22.9	10.1	0.227	2.49	1.08	0.231	0.109	0.107	1.01
shale	61.1	57.2	0.107	7.81	2.25	0.347	0.128	0.0393	3.26

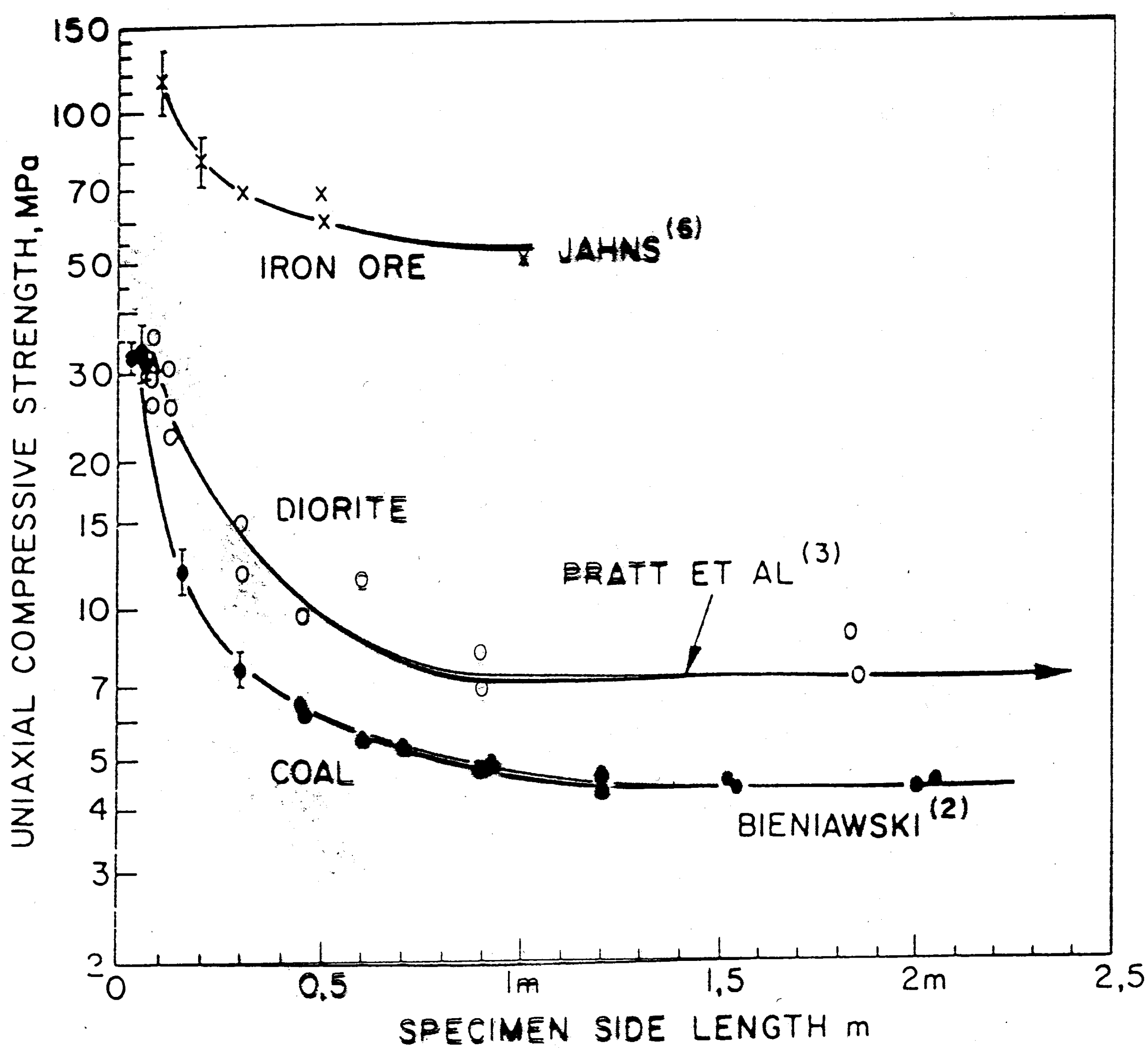
Relationship between critical strains of intact rocks and in-situ rock masses



Scale Effect

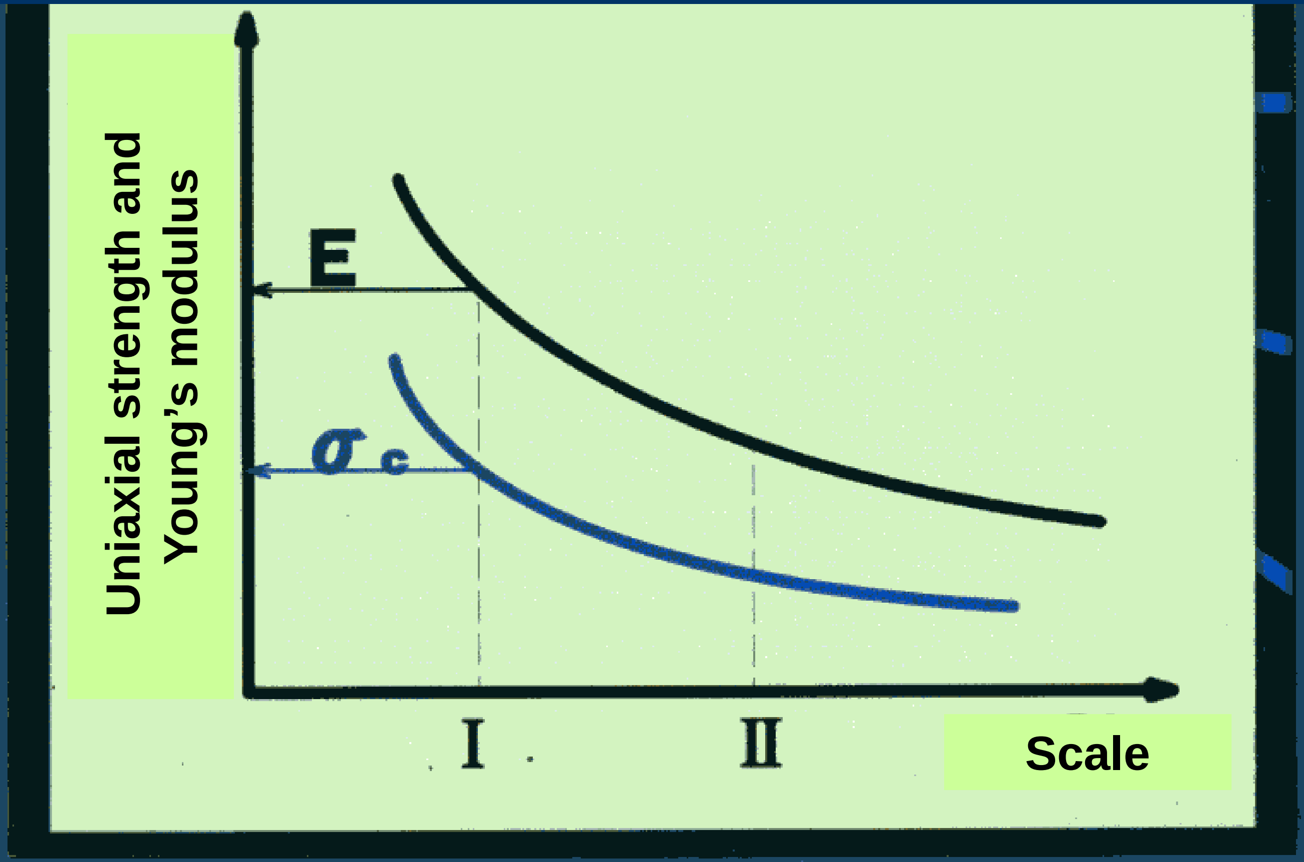


Scale Effect on Strength

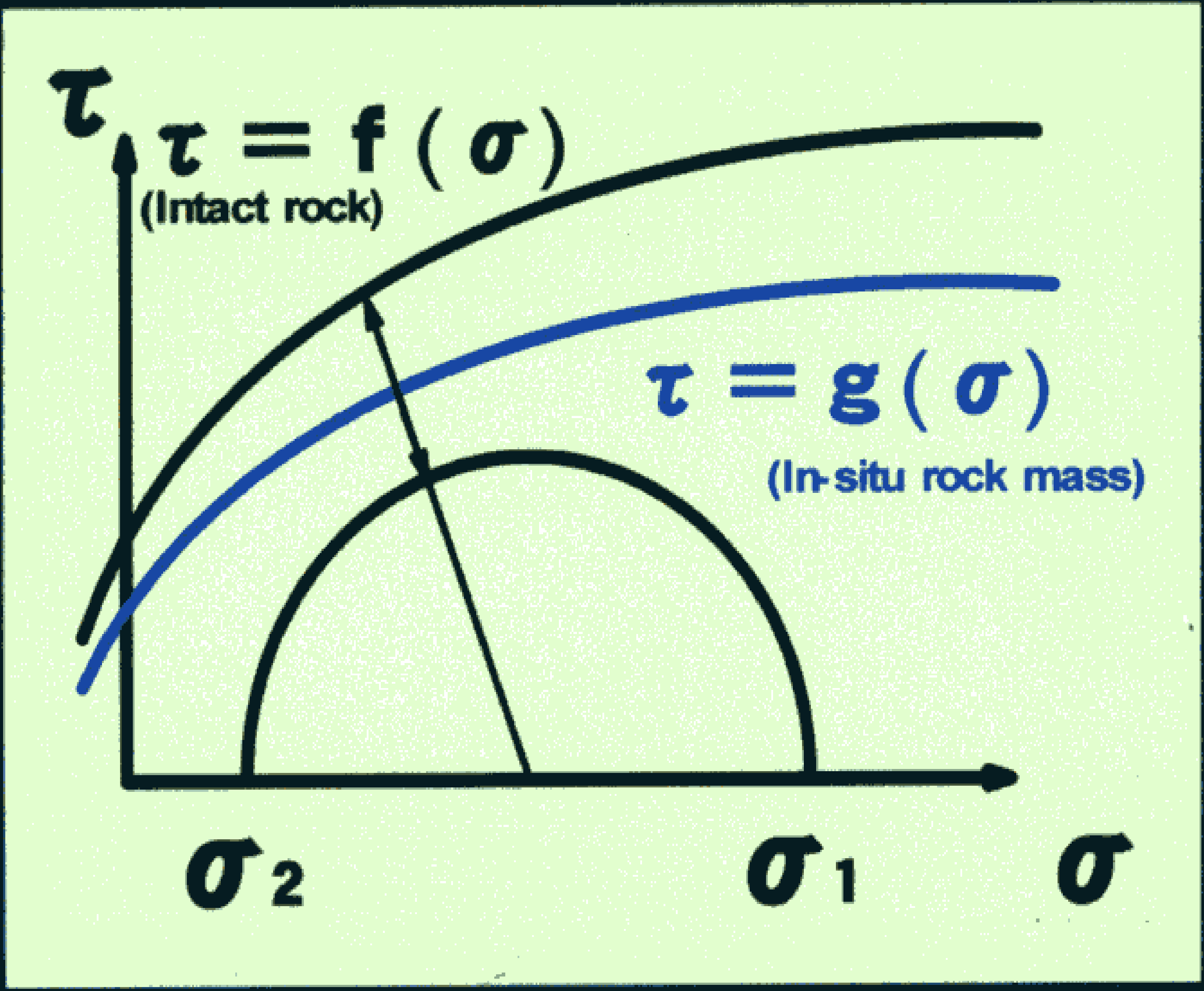


Scale effect of rock masses ;

Uniaxial compressive strength
and Young’s modulus



Failure Criteria



Hoek-Brown criterion (1)

The Hoek-Brown criterion is given as follows (Hoek and Brown, 1980a, 1980b, 1988, Hoek et al., 1992, Hoek, 1994),

$$\sigma_1' = \sigma_3' + \sigma_C\sqrt{m_b\frac{\sigma_3'}{\sigma_C} + s} \qquad \text{for} \quad GSI > 25 \qquad (1)$$

$$\sigma_1' = \sigma_3' + \sigma_C\left(m_b\frac{\sigma_3'}{\sigma_C}\right)^a \qquad \text{for} \quad GSI < 25 \qquad (2)$$

where σ_1' and σ_3' are major and minor principal effective stresses, respectively. σ_C is an uniaxial compressive strength of intact rock. m_b, s and a are material parameters depending on rock types, joint pattern, joint density, joint surface condition etc., which are given in the following formulas,

$$m_b = \exp\left(\frac{GSI - 100}{28}\right)m_i \qquad (3)$$

$$s = \exp\left(\frac{GSI - 100}{9}\right) \qquad (4)$$

$$a = 0.65 - \frac{GSI}{200} \qquad (5)$$

Hoek-Brown criterion (2)

Table 1 Generalised Hoek-Brown criterion (after Hoek, 1994)

GENERALISED HOEK-BROWN CRITERION

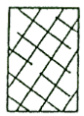
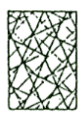
$$\sigma_1' = \sigma_3' + \sigma_c \left(m_b \frac{\sigma_3'}{\sigma_c} + s \right)^a$$

σ_1' = major principal effective stress at failure

σ_3' = minor principal effective stress at failure

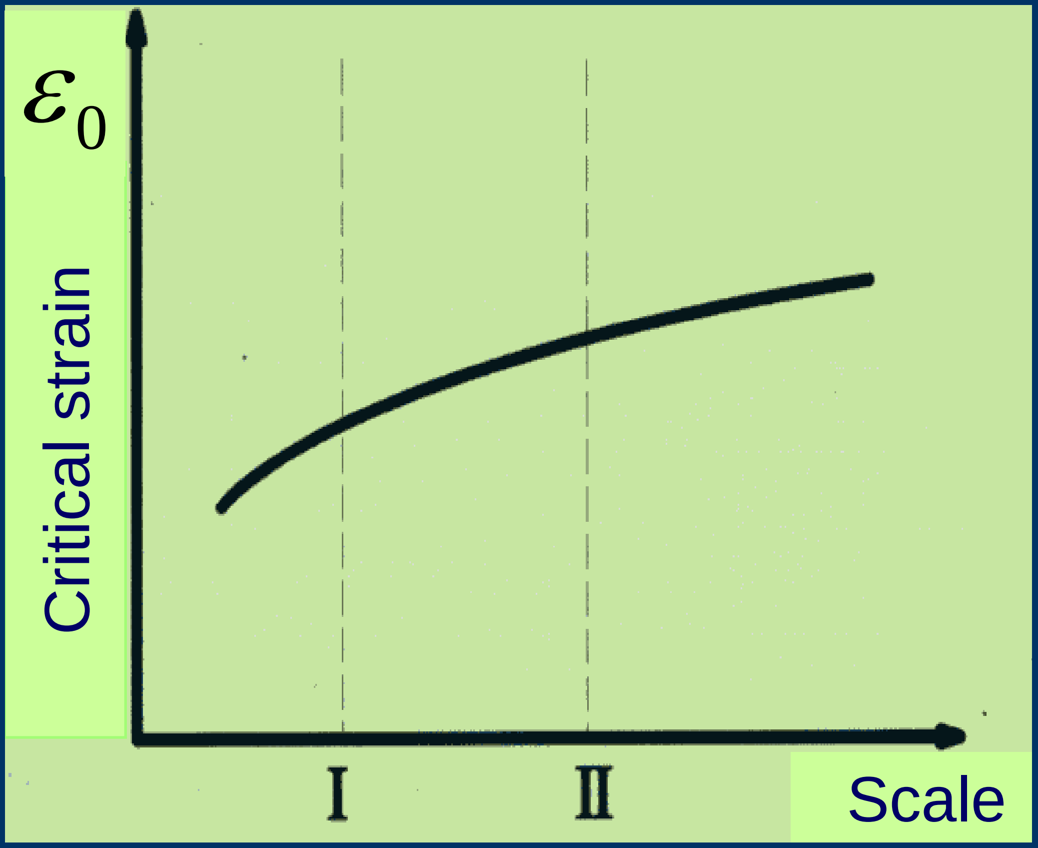
σ_c = uniaxial compressive strength of *intact* pieces of rock

m_b, s and a are constants which depend on the composition, structure and surface conditions of the rock mass

STRUCTURE	SURFACE CONDITION	VERY GOOD Very rough, unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered or altered surfaces	POOR Slickensided, highly weathered surfaces with compact coatings or fillings containing angular rock fragments	VERY POOR Slickensided, highly weathered surfaces with soft clay coatings or fillings
 BLOCKY - very well interlocked undisturbed rock mass consisting of cubical blocks formed by three orthogonal discontinuity sets	m_b/m_i s a E_m ν GSI	0.60 0.190 0.5 75,000 0.2 85	0.40 0.062 0.5 40,000 0.2 75	0.26 0.015 0.5 20,000 0.25 62	0.16 0.003 0.5 9,000 0.25 48	0.08 0.0004 0.5 3,000 0.25 34
 VERY BLOCKY - interlocked, partially disturbed rock mass with multifaceted angular blocks formed by four or more discontinuity sets	m_b/m_i s a E_m ν GSI	0.40 0.062 0.5 40,000 0.2 75	0.29 0.021 0.5 24,000 0.25 65	0.16 0.003 0.5 9,000 0.25 48	0.11 0.001 0.5 5,000 0.25 38	0.07 0 0.53 2,500 0.3 25
 BLOCKY/SEAMY - folded and faulted with many intersecting discontinuities forming angular blocks	m_b/m_i s a E_m ν GSI	0.24 0.012 0.5 18,000 0.25 60	0.17 0.004 0.5 10,000 0.25 50	0.12 0.001 0.5 6,000 0.25 40	0.08 0 0.5 3,000 0.3 30	0.06 0 0.55 2,000 0.3 20
 CRUSHED - poorly interlocked, heavily broken rock mass with a mixture of angular and rounded blocks	m_b/m_i s a E_m ν GSI	0.17 0.004 0.5 10,000 0.25 50	0.12 0.001 0.5 6,000 0.25 40	0.08 0 0.5 3,000 0.3 30	0.06 0 0.55 2,000 0.3 20	0.04 0 0.60 1,000 0.3 10

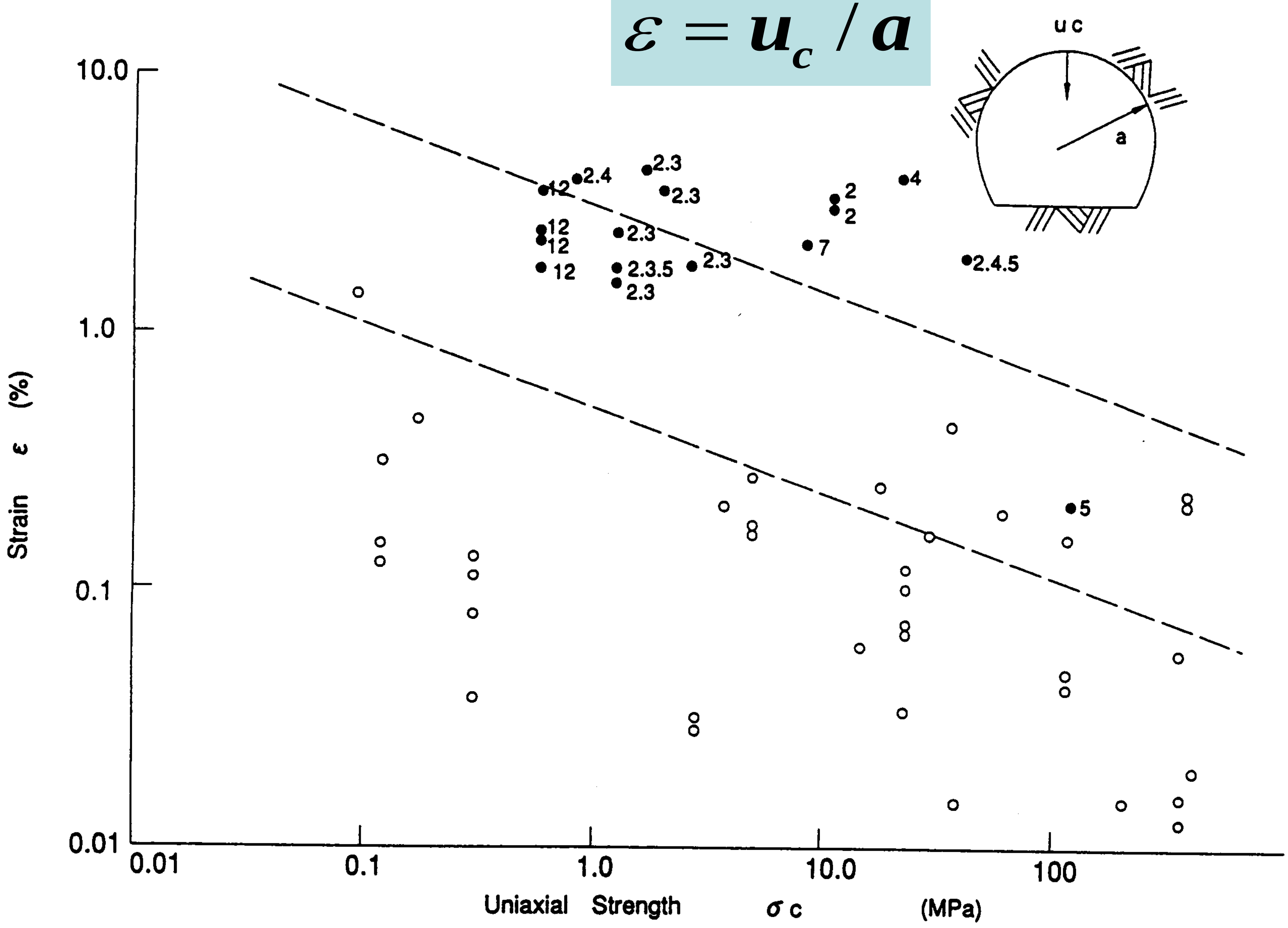
Note 1 Young's modulus E_m of in-situ rock masses is calculated from RMR. Unit of E_m are Mpa.

Scale effect of critical strain of rock masses



Relationship between critical strain and measured strain (obtained by crown settlements)

$$\varepsilon = u_c / a$$



Relationship between critical strain and measured strain (obtained by extensometers)

$$\varepsilon_r = \frac{u_1 - u_2}{\ell}$$

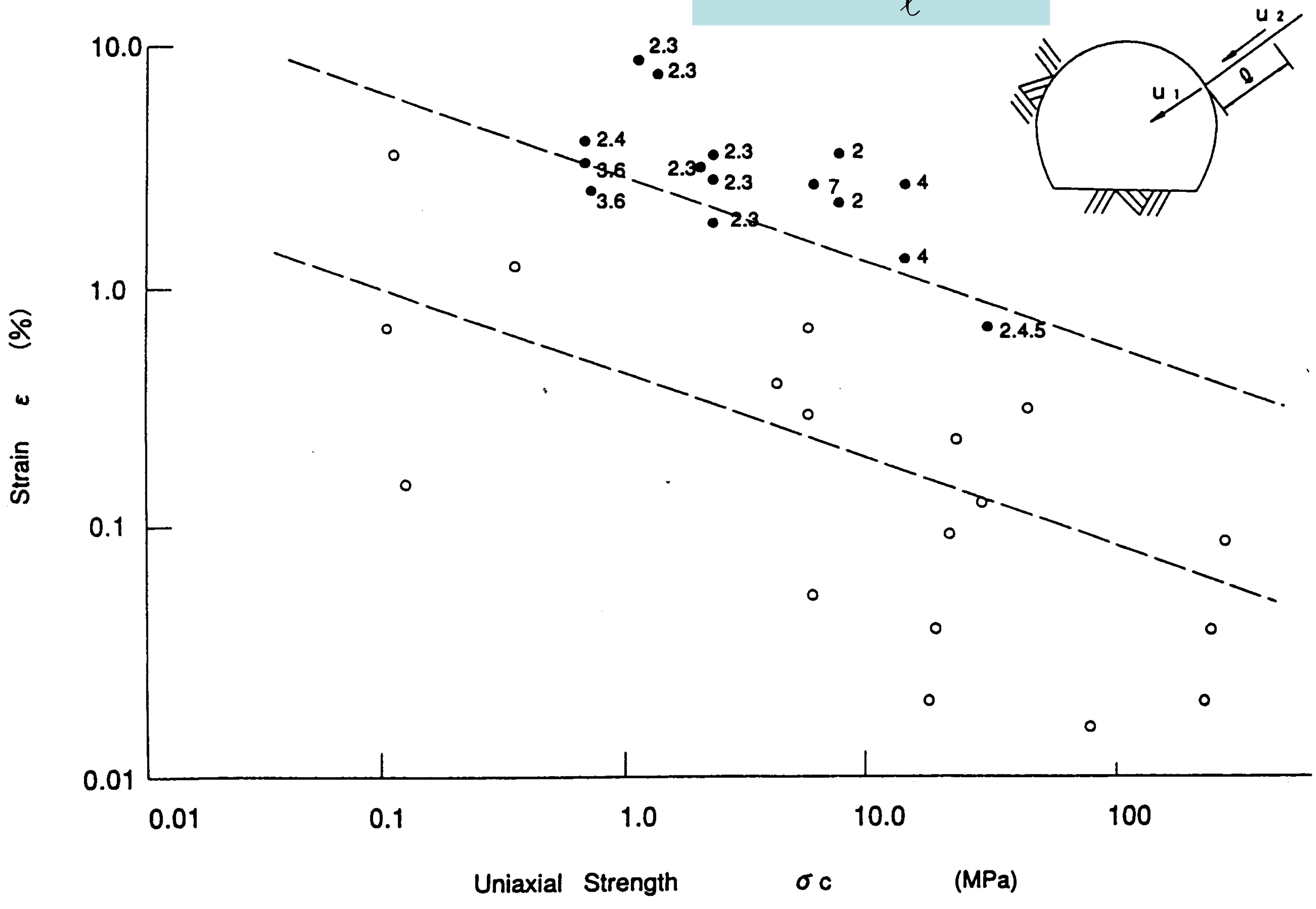
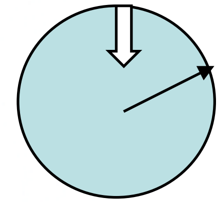


Table 2 Problems encountered during tunnelling

No.	Remarks
1	Difficulty in maintaining tunnel face
2	Failure or cracking in shotcrete
3	Buckling of steel ribs
4	Breakage of rock bolts
5	Fall-in of roof
6	Swelling at invert
7	Miscellaneous(unidentified)

After Prof. E. Hoek



$$\varepsilon = u_c / a$$

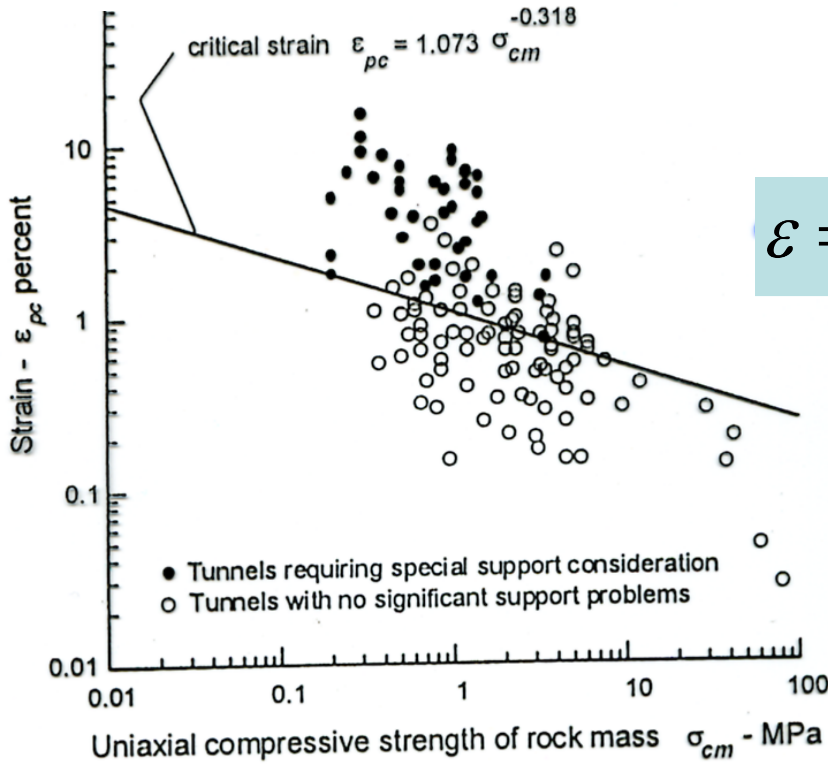


Figure 7: Percentage strain for different rock mass strengths. The points plotted are for the Second Freeway, the Pinglin and the New Tienlun Headrace tunnels in Taiwan.

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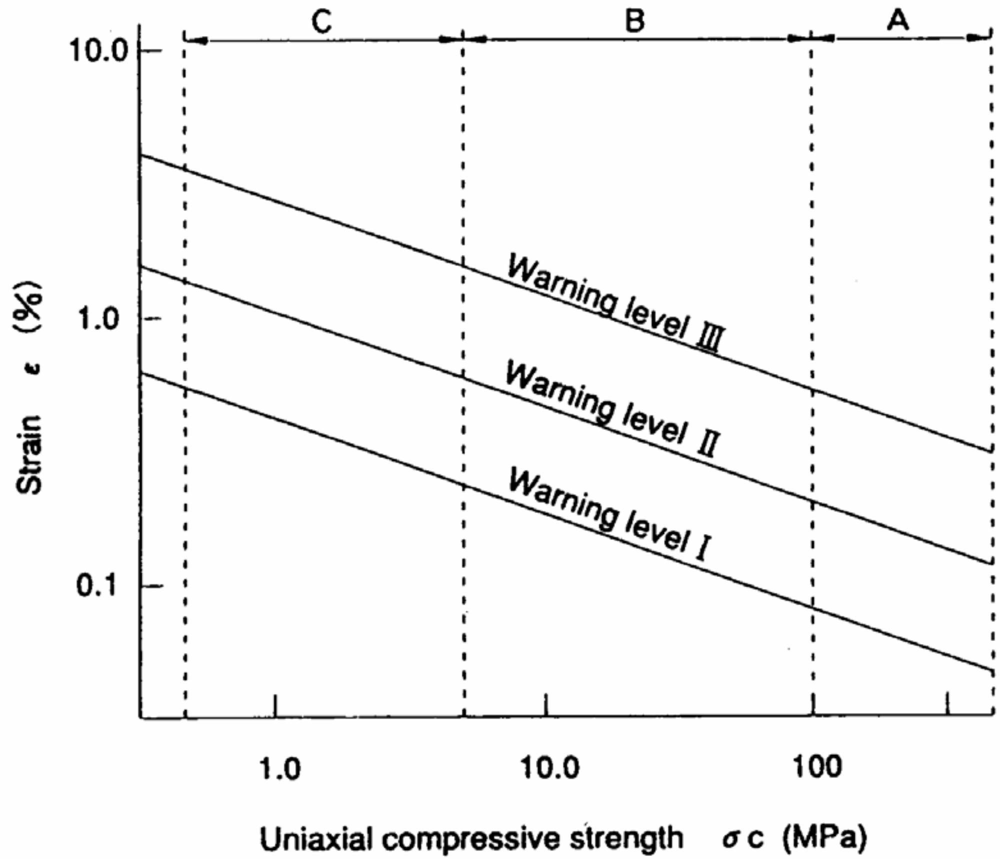
[1] Information in this plot was supplied by Dr J.C. Chern of Sinotech Engineering Consultants Inc., Taipei.

Hazard warning levels for assessing the stability of tunnels

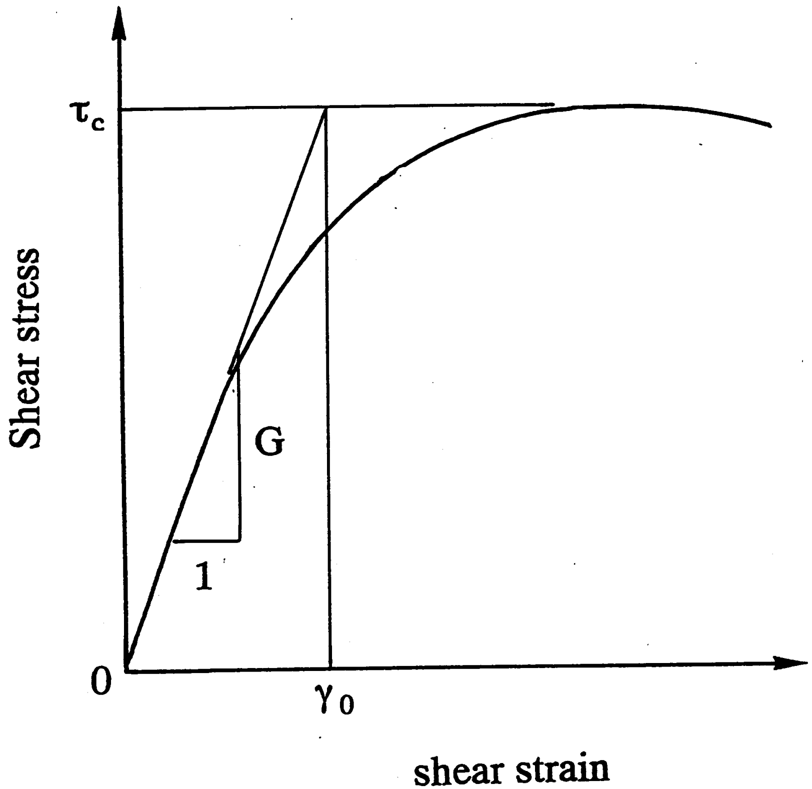


	A	B	C
Ⅰ	0.3~0.5	0.5~1	1~3
Ⅱ	1~1.5	1.5~4	4~9
Ⅲ	3~4	4~11	11~27

(Unit : cm)
(Radius of tunnel: 5.00m)

In order to assess the stability of tunnels more precisely, the critical shear strain was proposed.

Definition of critical shear strain



Critical shear strain γ_0

$$\gamma_0 = \tau_c / G$$

where τ_c : ***shear strength***

G : ***shear modulus***

Stability assessment of tunnels

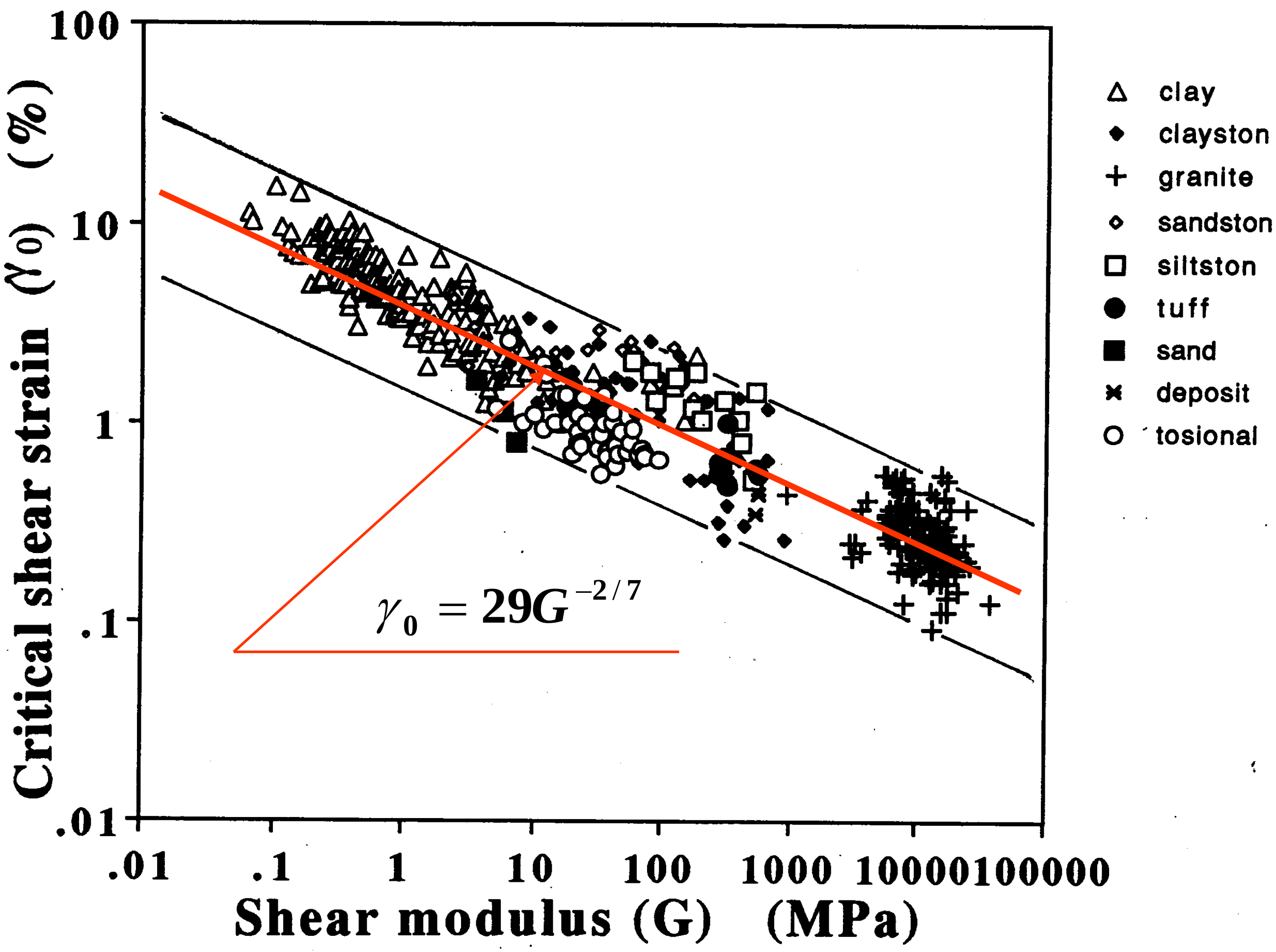
The maximum shear strain should be smaller than the critical shear strain.

$$\gamma_{\max} < \gamma_0 \quad \text{------------ } \textit{Stable}$$

If the maximum shear strain becomes larger than the critical shear strain, tunnel support measures must be installed so as to keep the maximum shear strain remain being less than the critical shear strain.

$$\gamma_{\max} > \gamma_0 \quad \text{----------- } \textit{Unstable}$$

Laboratory experimental results
(Relationship between critical shear strain and shear modulus)



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- Observational method in tunnels (Hazard Warning Level)
- Back analysis
- Anisotropic parameter
- Observational method in slopes
- Case study (Tunnel)
- Case study (Slope)
- Conclusions

Definition of back analysis

Input data

- External Forces
- Young's modulus
- Poisson's ratio
- Strength parameters (c,Φ)
- Boundary conditions
- . . . .

Output results

→ **Forward Analysis** →

Output results

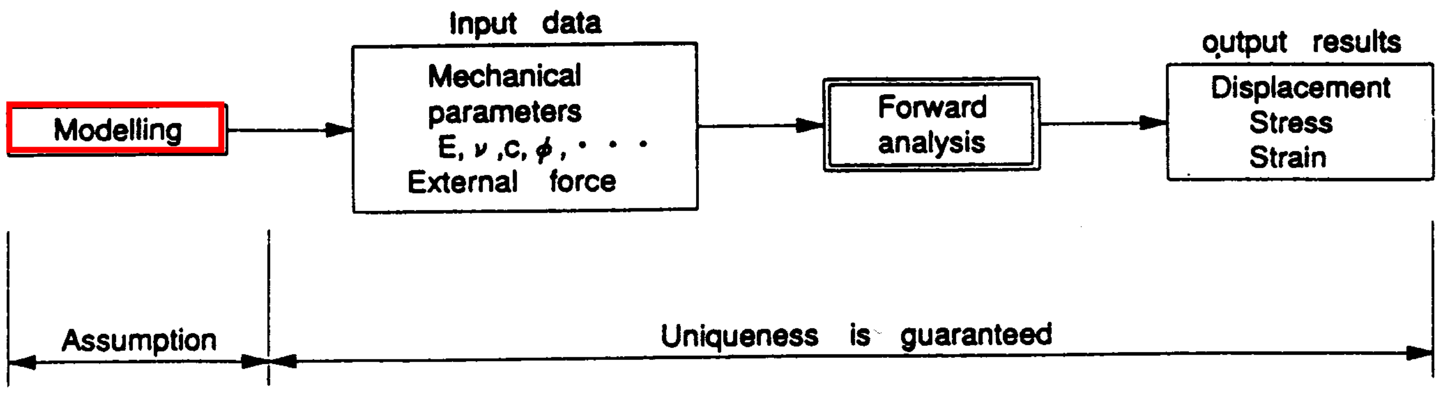
- Stress
- Strain
- Displacements
- . . . .
- . . . . . .

Input data

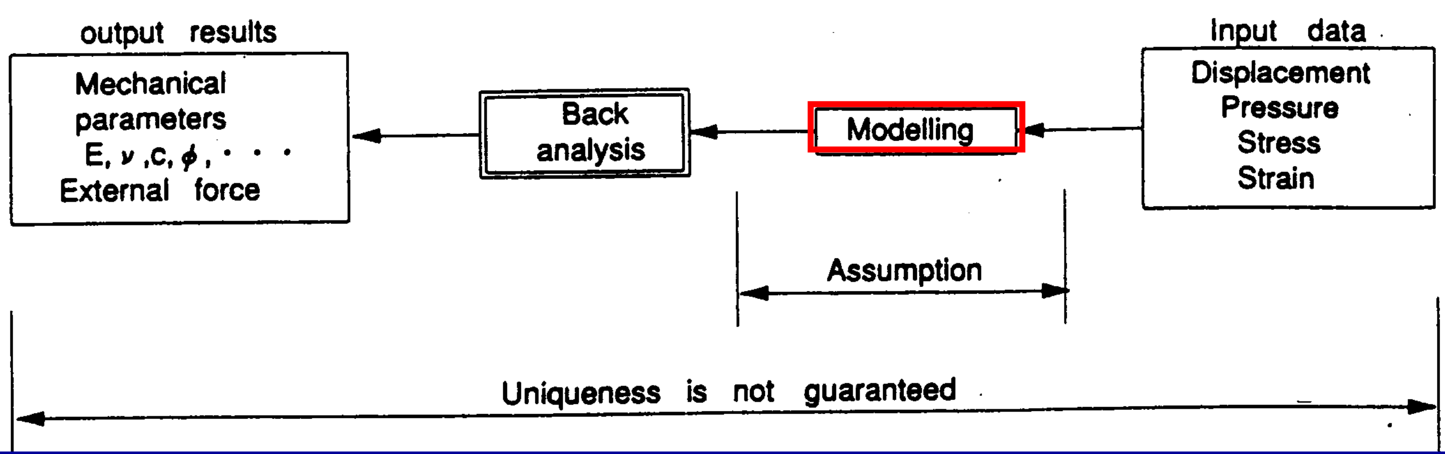
← **Back Analysis** ←

Modeling in forward analysis and back analysis

(1) Forward analysis



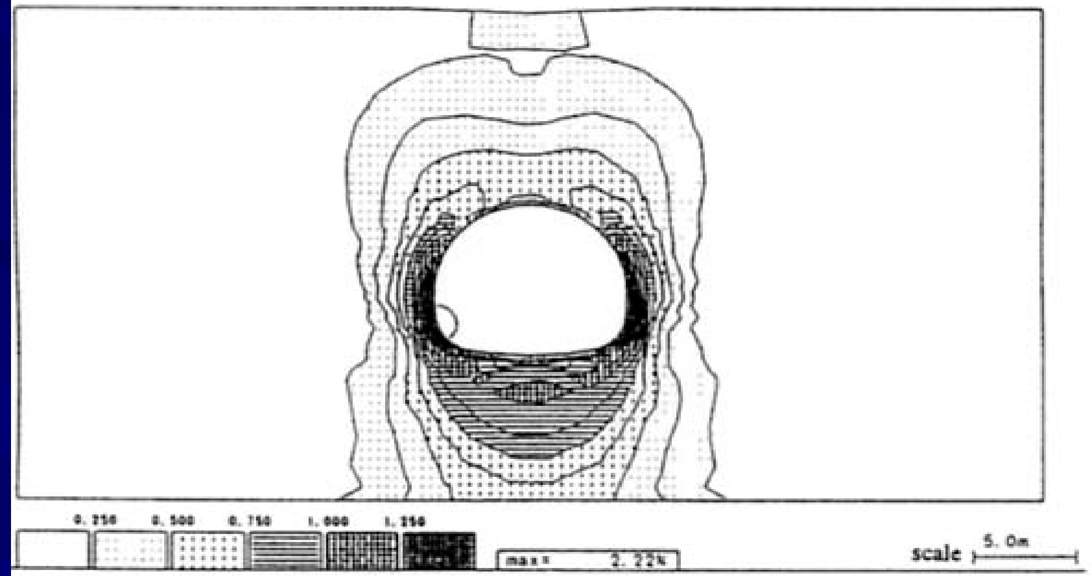
(2) Back analysis



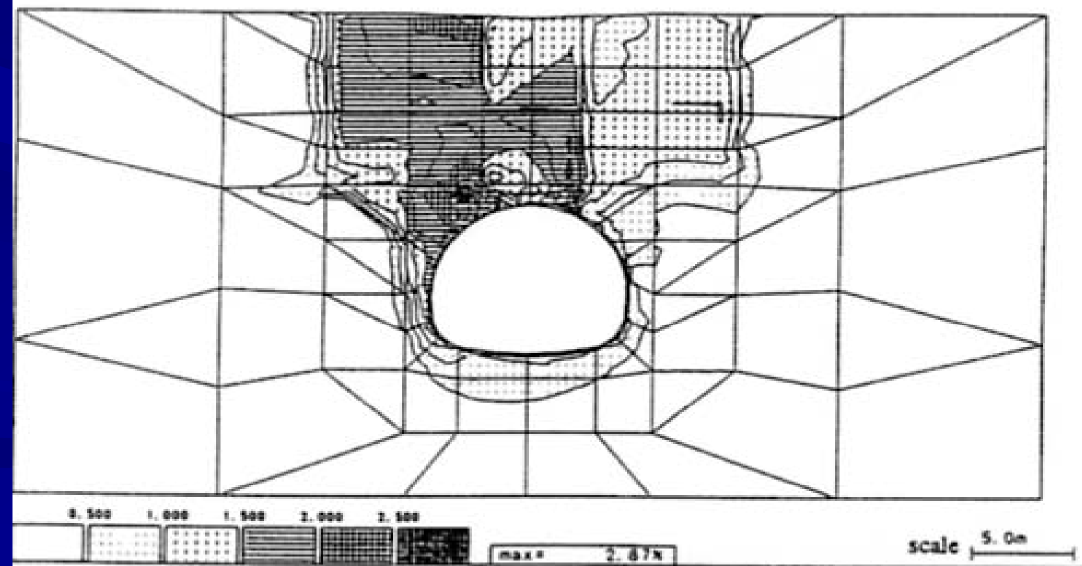
Numerical Modeling in Back Analysis

- In back analysis, it is extremely important that a mechanical model should not be assumed, but it should be identified by a back analysis

Maximum shear strain distribution obtained by a back analysis (assuming an isotropic elastic model) using the same measured displacements



Maximum shear strain distribution (Isotropic elastic model)



Maximum strain distribution (No model is assumed)

Numerical modeling of tunnel
Modulus of deformability of tunnel lining and surrounding media

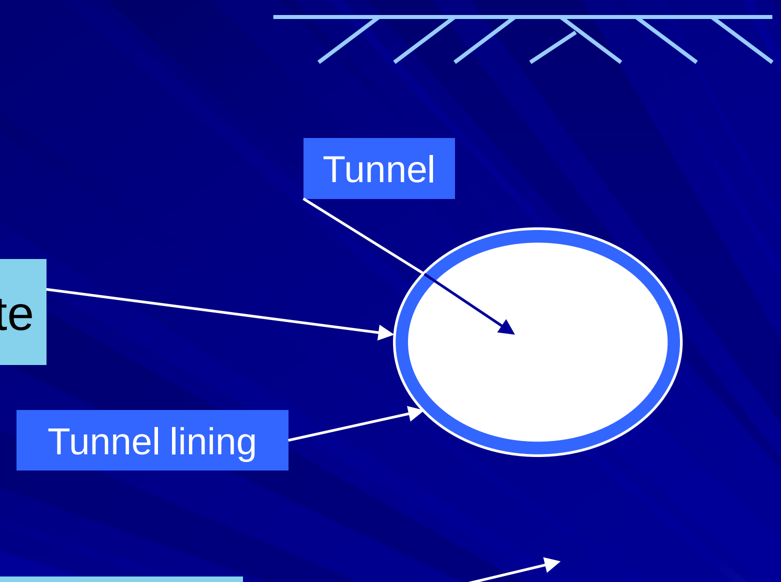
- At the design stage of tunnels we use,

  for shotcrete

  $E_\ell =$ Modulus of deformability of concrete

  for surrounding media

  $E_g =$ Modulus of deformability of soils and/or rocks



- In general

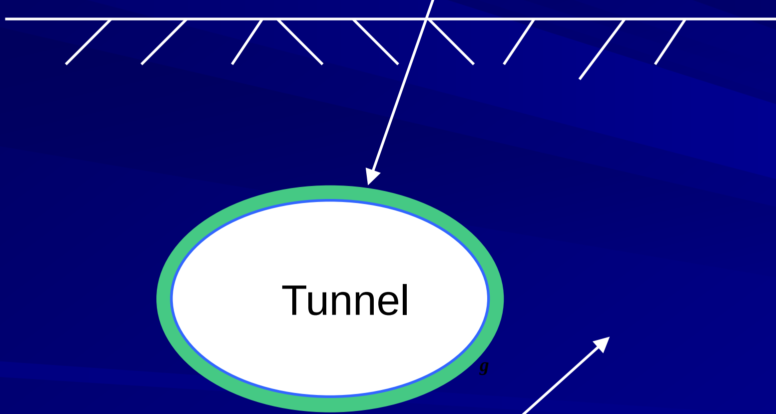
$$E_\ell / E_g > 1$$

The results of back analysis

Shallow tunnel excavated in sandy diluvial formation

Modulus of deformability of tunnel lining consisting of shotcrete and steel ribs

$$E_\ell$$



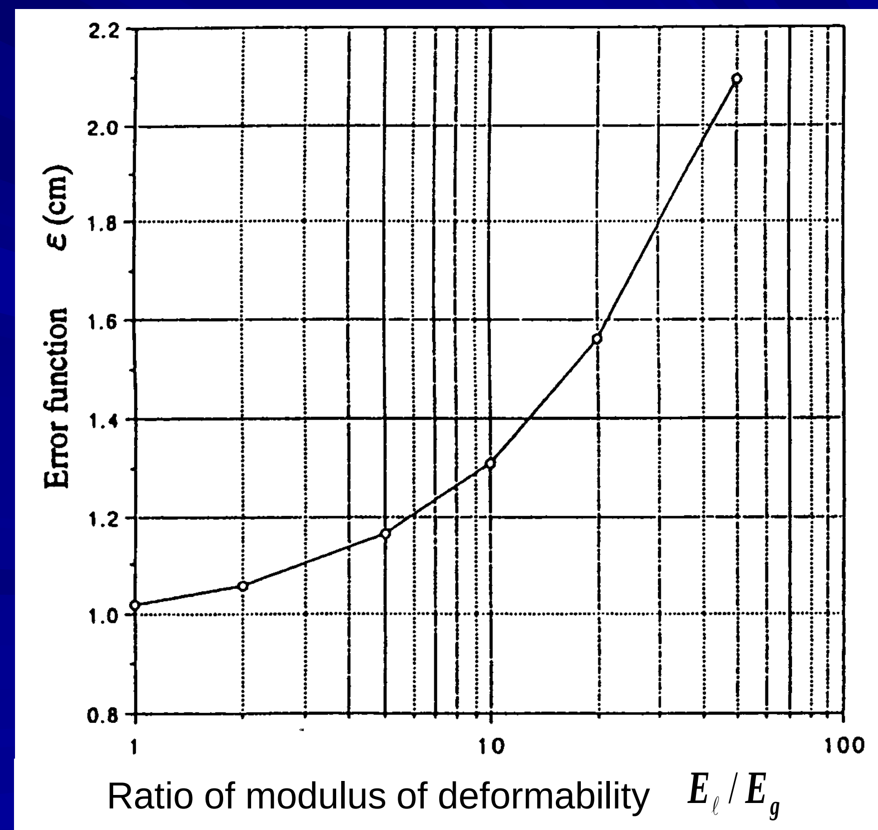
Modulus of deformability of the ground E_g

Error function

$$\varepsilon = \frac{\sum_{i=1}^{N}(u_i^m - u_i^c)^2}{\sum_{i=1}^{N} u_i^m}$$

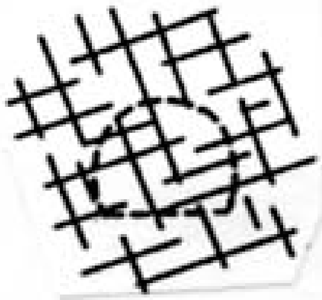
u_i^m : Measured displacements

u_i^c : Computed displacements



Conventional procedure for modeling of jointed rock masses with rock bolts

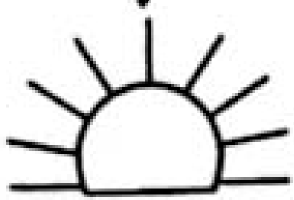
(Continuum approach)



Modeling of jointed rock masses as an equivalent continuum

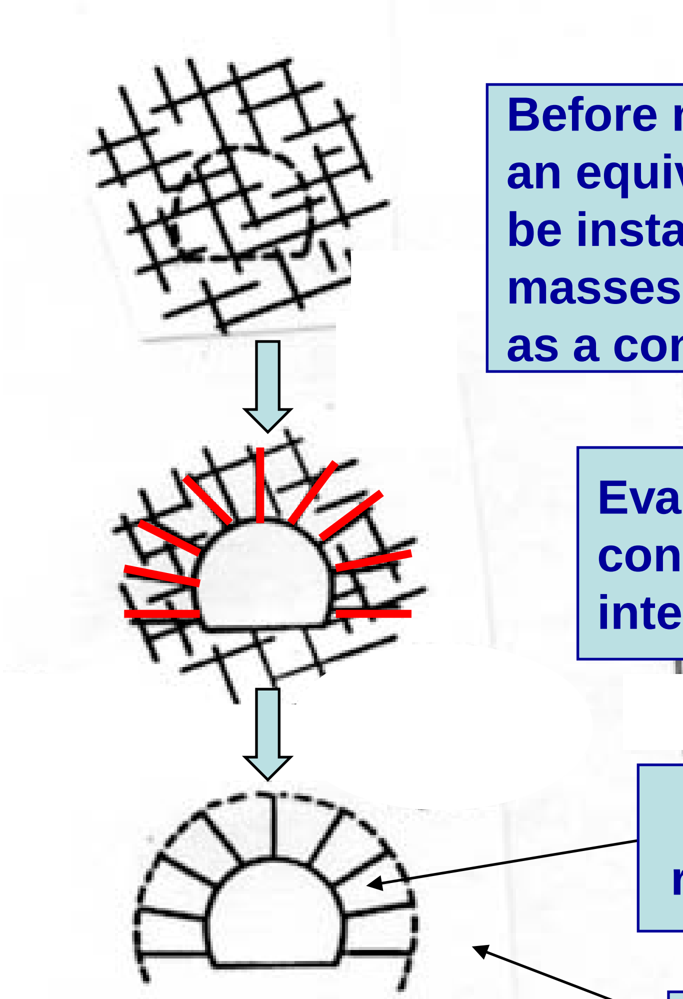
$E, \nu, c, \phi ......$

Evaluation of mechanical parameters of the equivalent continuum



Modeling of tunnel excavation with installation of rock bolts considering excavation sequence

Modeling of jointed rock masses reinforced by rock bolt (Continuum approach for hard rock type)



Before modeling jointed rock masses as an equivalent continuum, rock bolts should be installed. After that the jointed rock masses reinforced by rock bolts are modeled as a continuum.

Evaluation of material parameters of the continuum considering the effect of interaction between joints and rock bolts

E^*, ν^*, c^*, ϕ^* **for the region reinforced by rock bolts**

E, ν, c, ϕ **for far field**

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Conventional elasto-plastic analysis

Numerical algorithm for conventional elasto-plastic analysis has been developed for forward analysis.
So, it is too "stiff" to apply for back analysis.

In back analysis numerical algorithm should be "flexible" enough to obtain a good agreement between measurements and computations.

Numerical algorithm for conventional elasto-plastic analysis

Flow rule

$$\{d\varepsilon\} = \{d\varepsilon_e\} + \{d\varepsilon_p\}$$

$$\{d\varepsilon_e\} = [D_e]^{-1}\{d\sigma\}$$

$$\{d\varepsilon_p\} = \lambda \frac{\partial Q}{\partial \{\sigma\}}$$

Q **: plastic potential function**

Yielding function

$$F(\{\sigma\}, k) = 0$$

$$\therefore$$

$$\{d\sigma\} = [D_{ep}]\{d\varepsilon\}$$

where

$$[D_{ep}] = [D_e] - \frac{[D_e]\left\{\frac{\partial Q}{\partial \{\sigma\}}\right\}\left\{\frac{\partial F}{\partial \{\sigma\}}\right\}^T [D_e]}{\left\{\frac{\partial F}{\partial \{\sigma\}}\right\}^T [D_e]\left\{\frac{\partial Q}{\partial \{\sigma\}}\right\} - A}$$

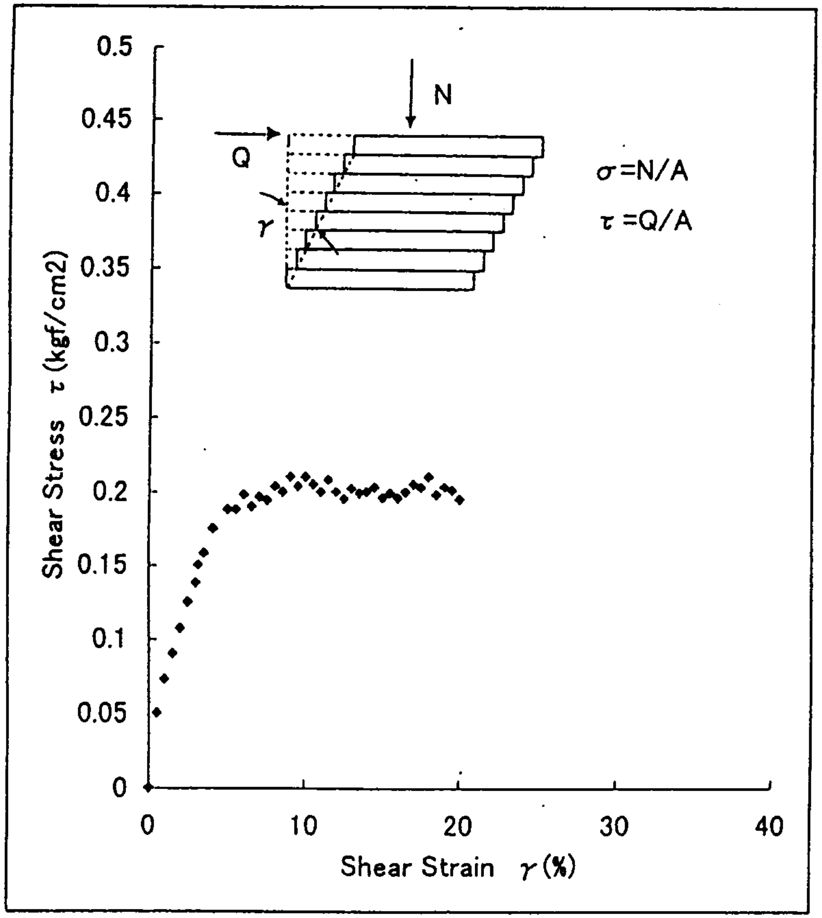
$F = Q$ ····· **Associated flow rule**

$F \neq Q$ ····· **Non-associated flow rule**

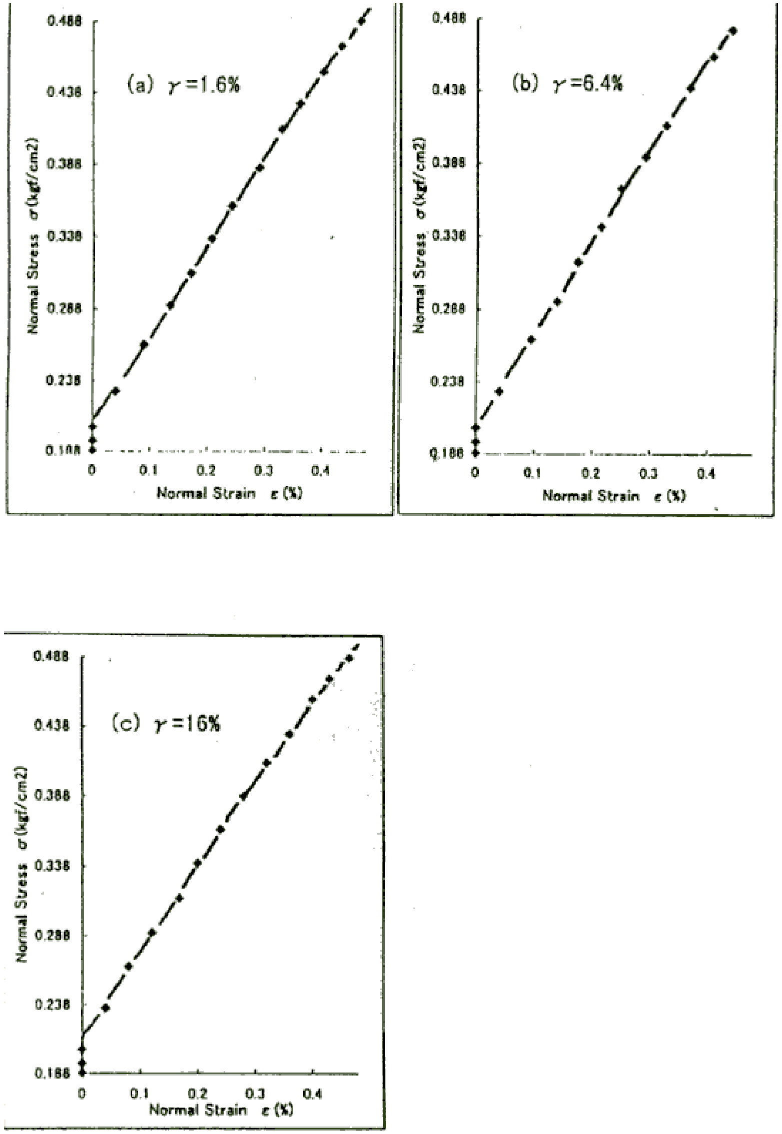
Back analysis in geotechnical engineering

In back analysis a constitutive equation should be simple enough to back-analyze its mechanical parameters uniquely from measured displacements.

Simple Shear Test



Normal stress versus normal strain during a simple shear test



Shear modulus & Young's modulus

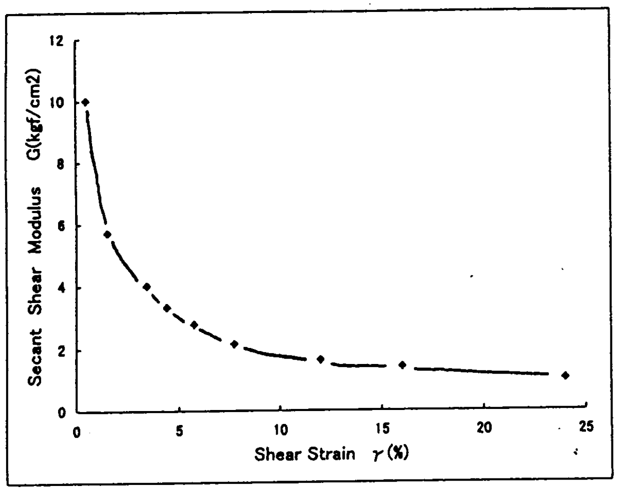
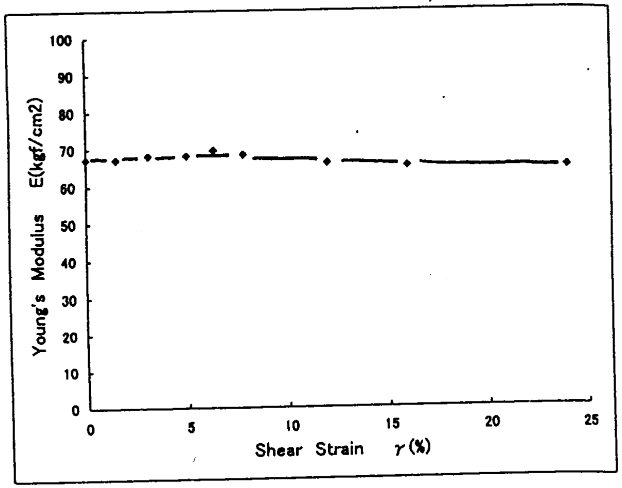


Fig.3 Shear modulus versus shear strain.



Young's modulus and Poisson's ratio for non-elastic materials

Young's modulus and Poisson's ratio proposed by Duncan and Chang

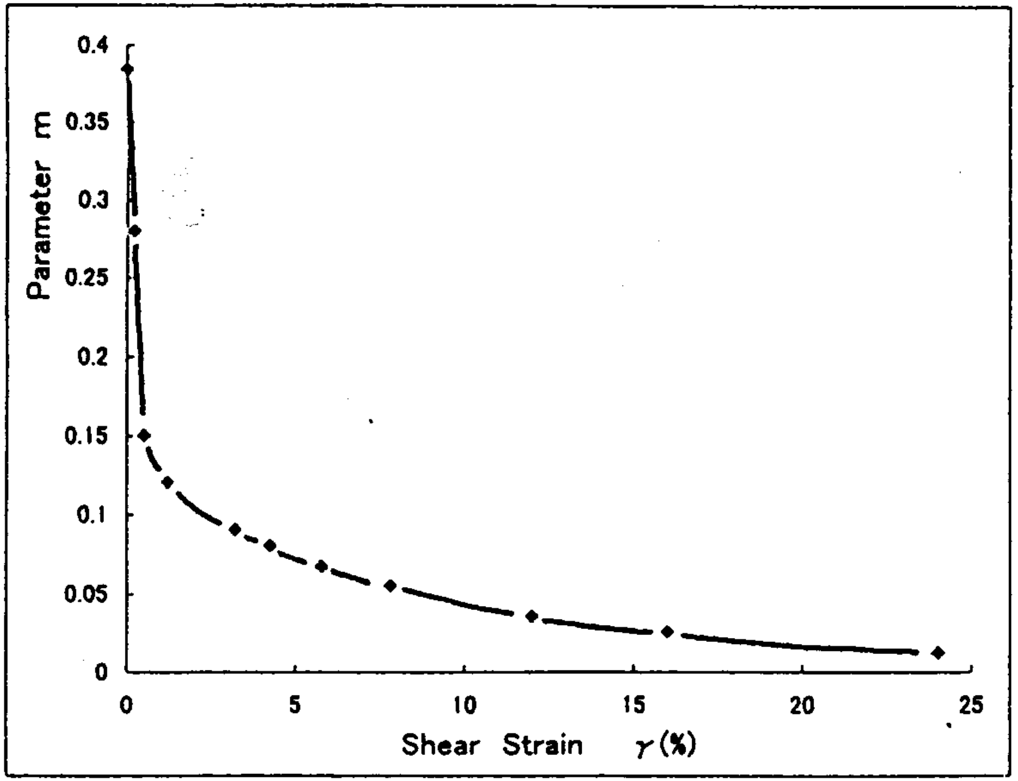
$$E = E_i \left[1 - \frac{R_f(\sigma_1 - \sigma_3)(1 - \sin\phi)}{2\sigma_3 \sin\phi + 2c\cos\phi} \right]^2$$

$$\nu = \nu_i / (1 - A)^2$$

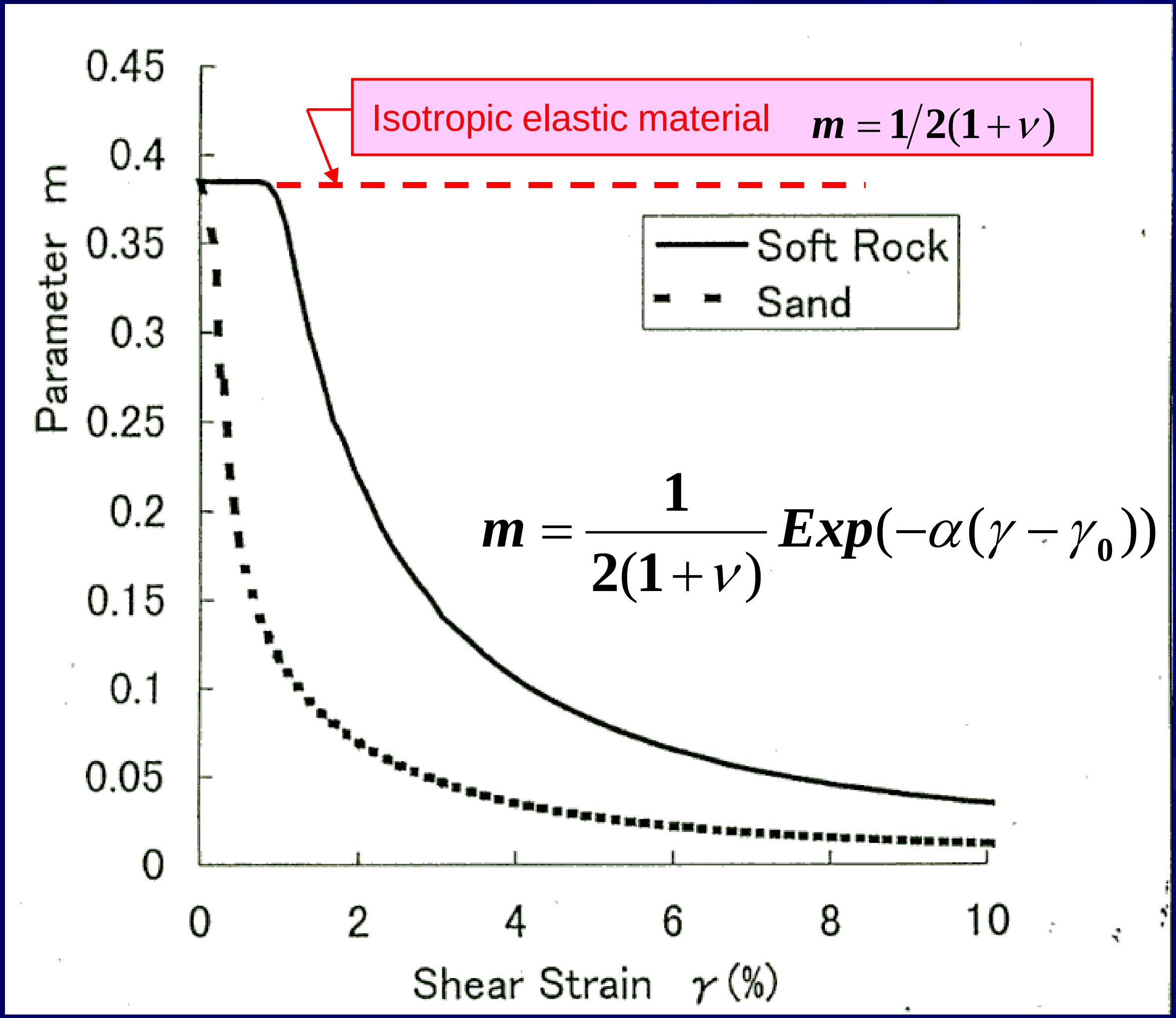
where

E_i : Initial Young's modulus
c : Cohesion
ϕ : Internal friction angle
R_f : Coefficient representing non-linearity
ν_i : Initial Poisson's ratio
A : Coefficient depending on stress

Anisotropic parameter *m* (= G/E)



Anisotropic parameter *m* for different materials as a function of shear strain



Stress and strain relationship in plane strain condition

$$\{\sigma\} = [\boldsymbol{D}]\{\varepsilon\}$$

Where

$$[\boldsymbol{D}] = \frac{\boldsymbol{E}}{\mathbf{1} - \nu - \mathbf{2}\nu^{\mathbf{2}}} \begin{bmatrix} \mathbf{1} - \nu & \nu & \mathbf{0} \\ \nu & \mathbf{1} - \nu & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & \boldsymbol{m}(\mathbf{1} - \nu - \mathbf{2}\nu^{\mathbf{2}}) \end{bmatrix}$$

E : Young's modulus

ν : Poisson's ratio

$$\boldsymbol{m} = \mathbf{1}/\mathbf{2}(\mathbf{1} + \nu) - \boldsymbol{d}$$

$\boldsymbol{d}$: Anisotropic damage parameter

Anisotropic damage parameter *d*

$$d = 1/2(1+\nu) - m$$

where ν : *Poisson's ratio*

m : *Ratio of shear modulus to Young's modulus on a slip plane*

$$d = f(\gamma)$$

$$f(\gamma) = 0 \qquad \text{for } \gamma \le \gamma$$

$$f(\gamma) = \left(\frac{1}{2(1+\nu)} - \beta \right) \left[1 - Exp\left\{ -\alpha(\quad - \quad \right\} \right] \qquad \text{for } \gamma > \gamma$$

where α, β : *matrial constants*

γ_0 : *critical shear strain*

Numerical simulation

for expressing stress-strain relationship for strain-hardening, perfect plastic and strain-softening behaviours of materials

Two-dimensional plane strain condition

- Bi-axial compression-

$=$

$=$

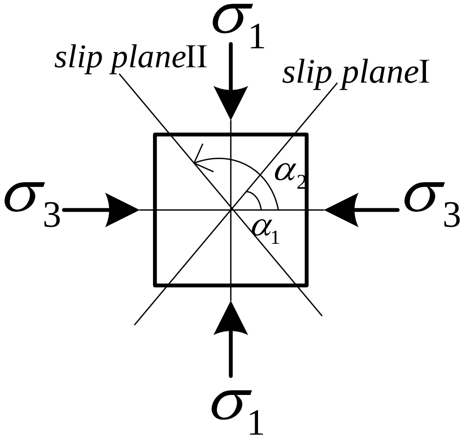
$=$

$=$

$=1.34\%$

$= \quad {}^{\circ}$

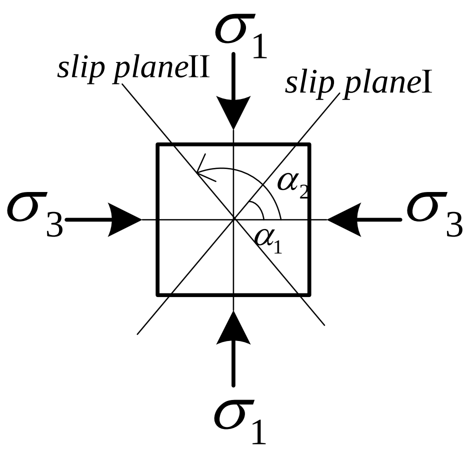
$= \quad =$



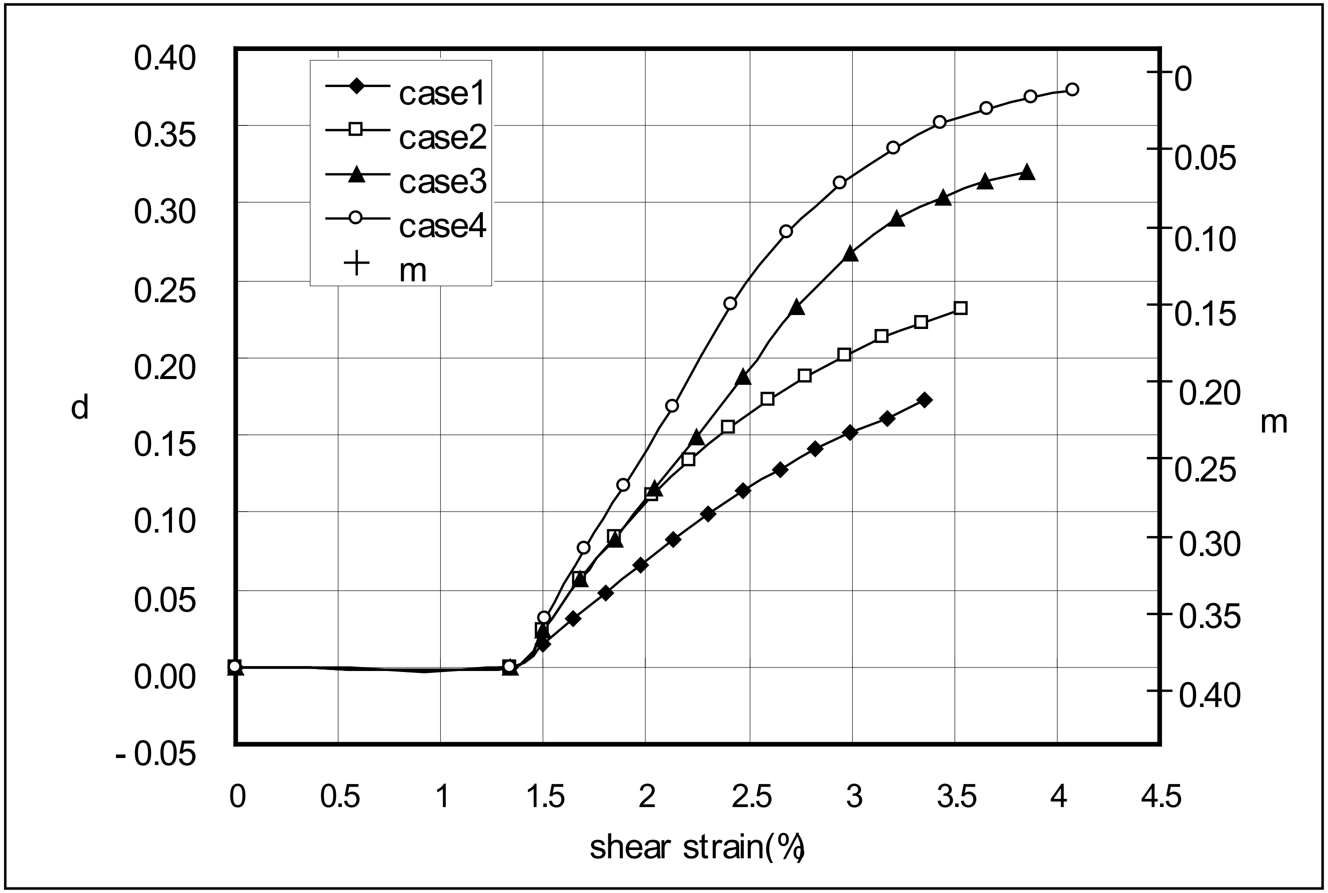
$$\alpha_1 = 55^{\circ}$$

$$\alpha_2 = 125^{\circ}$$

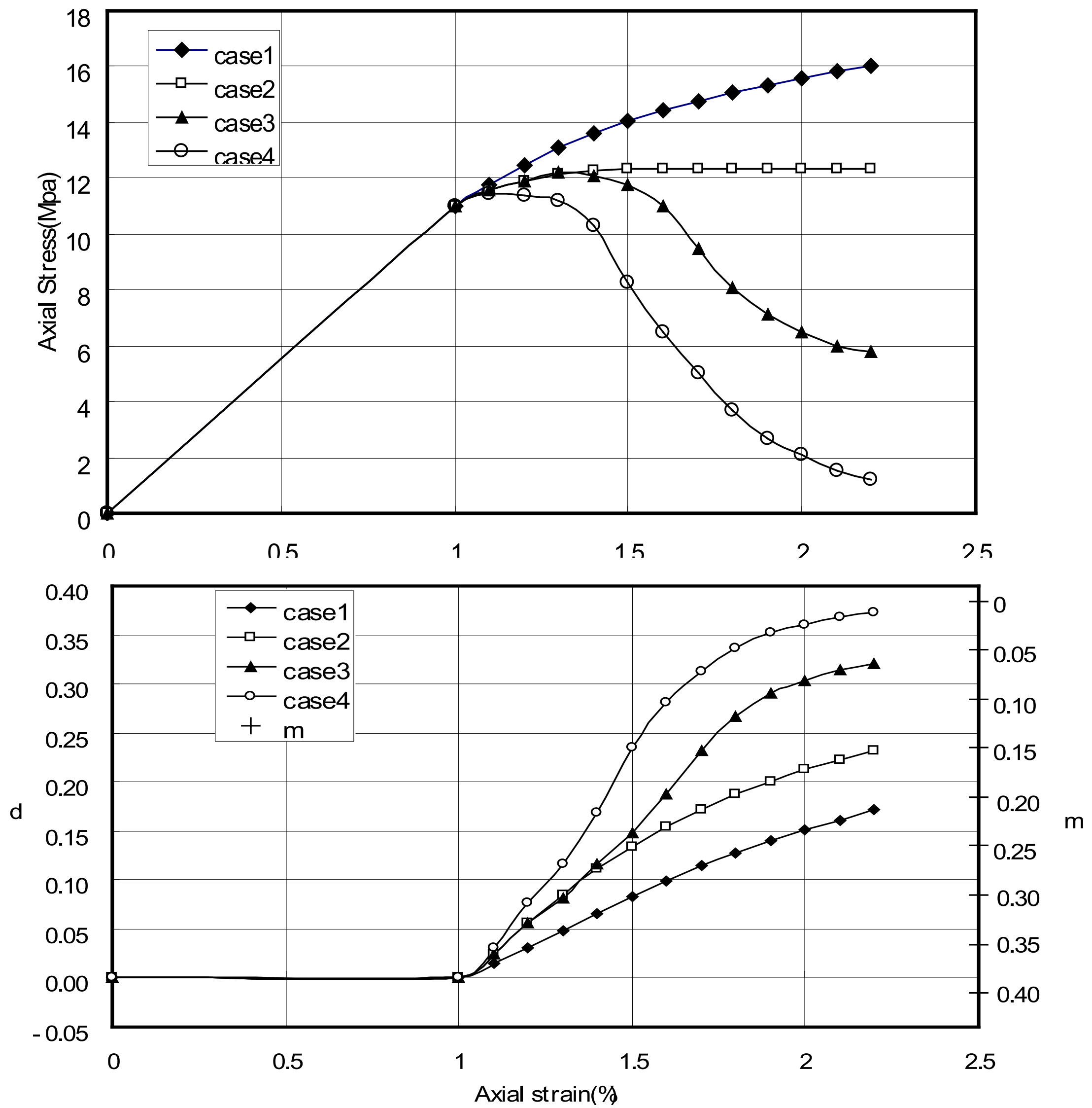
Parameters *d* and *m* given as a function of shear strain on a conjugate slip plane



$$\sigma_3 = 0$$



Stress-strain relationship with respect to parameters *d* and *m*



Note

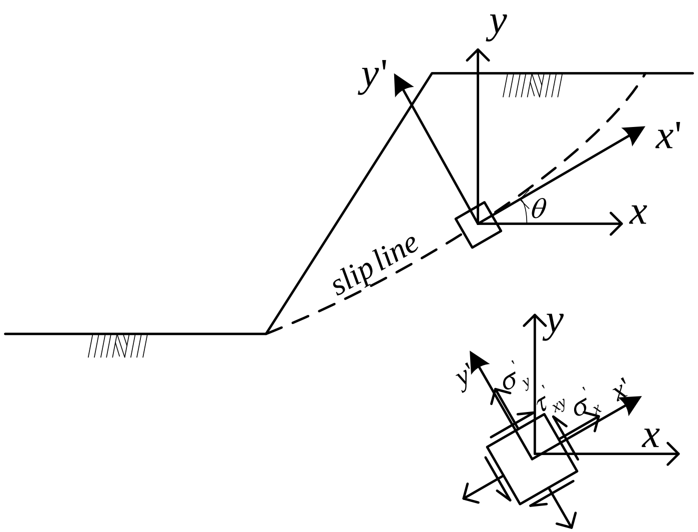
- Anisotropic parameter *m* is a simple monotonic increase function of shear strain.

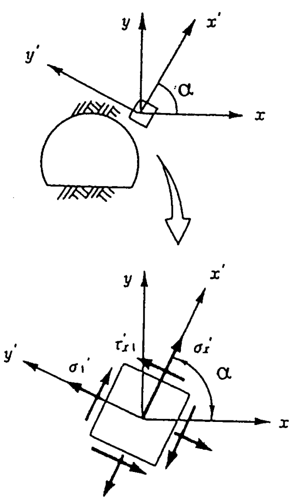
- Introducing the anisotropic parameter *m*, the following three different types of nonlinear stress-strain relationship of materials can be easily simulated, that is,

  (1) Strain-hardening
  (2) Ideal plastic
  (3) Strain softening

- The parameter *m* can be determined by back analysis of field measurement as well as laboratory experiments.

Stress-strain relationship in local and global coordinates





In local coordinate (plane strain condition),

$$\{\sigma'\} = [D']\{\varepsilon'\}$$

where

$$[D'] = \frac{E}{1-\nu-2\nu^2}\begin{bmatrix} 1-\nu & \nu & c_1 \\ \nu & 1-\nu & c_2 \\ c_1 & c_2 & m(1-\nu-2\nu^2) \end{bmatrix}$$

where

E : *Young's modulus*

ν : *Poisson's ratio*

c_1, c_2 : *dilatancy parameters*

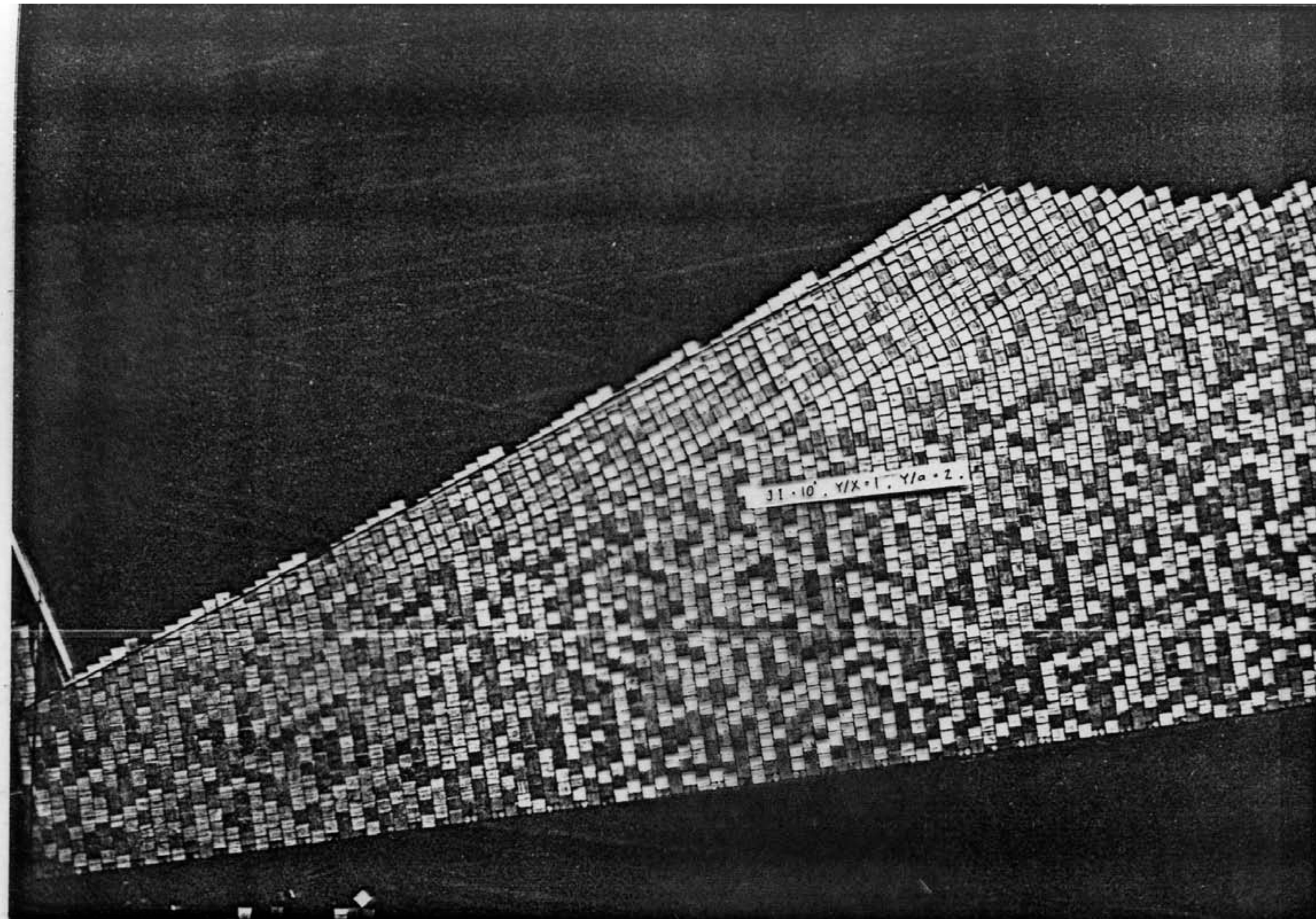
$m = 1/2(1+\nu) - d$

In global coordinate, $[D]$ *is expressed as,*

$$[D] = [T][D'][T]^T$$

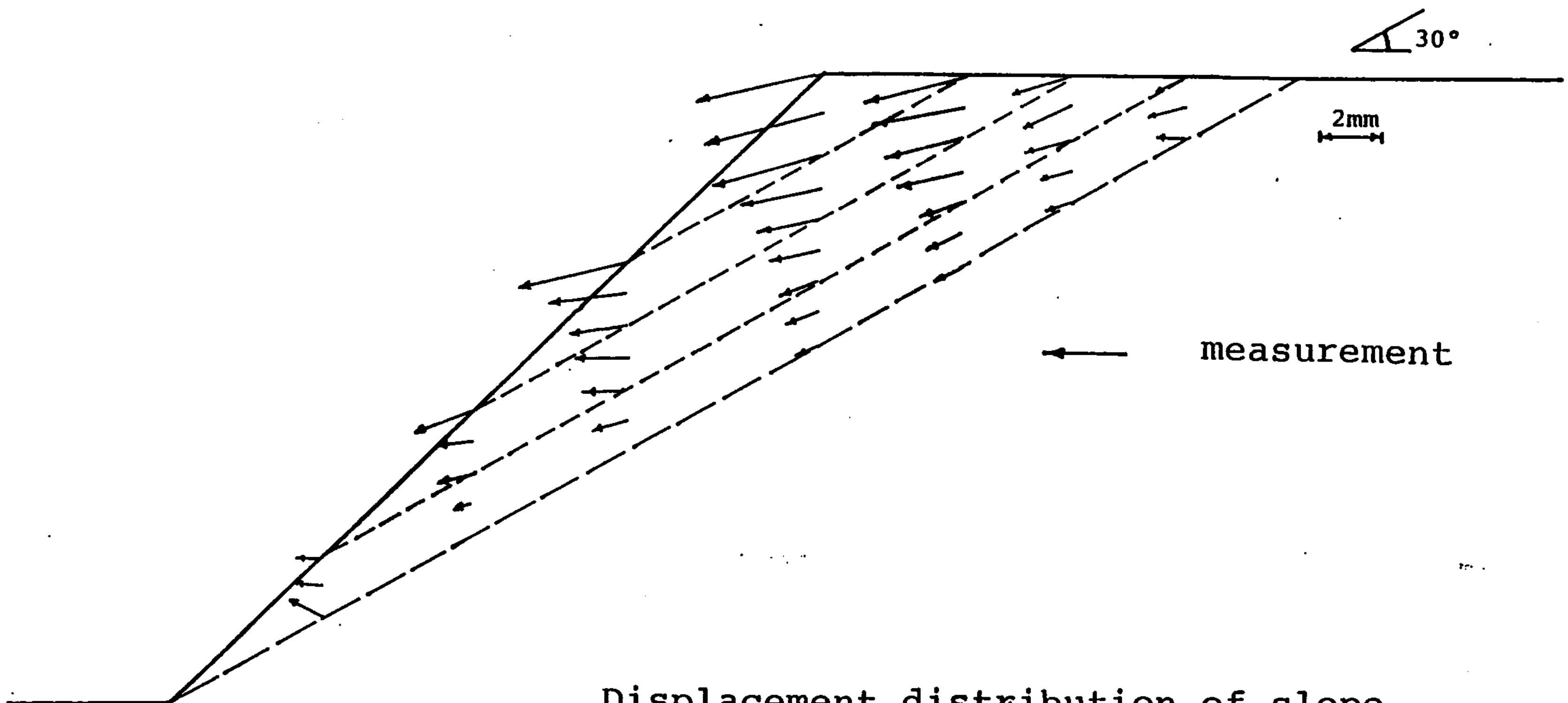
where $[T]$: *transformation matrix*

Laboratory experiment for toppling in slope excavation (aluminum bars)



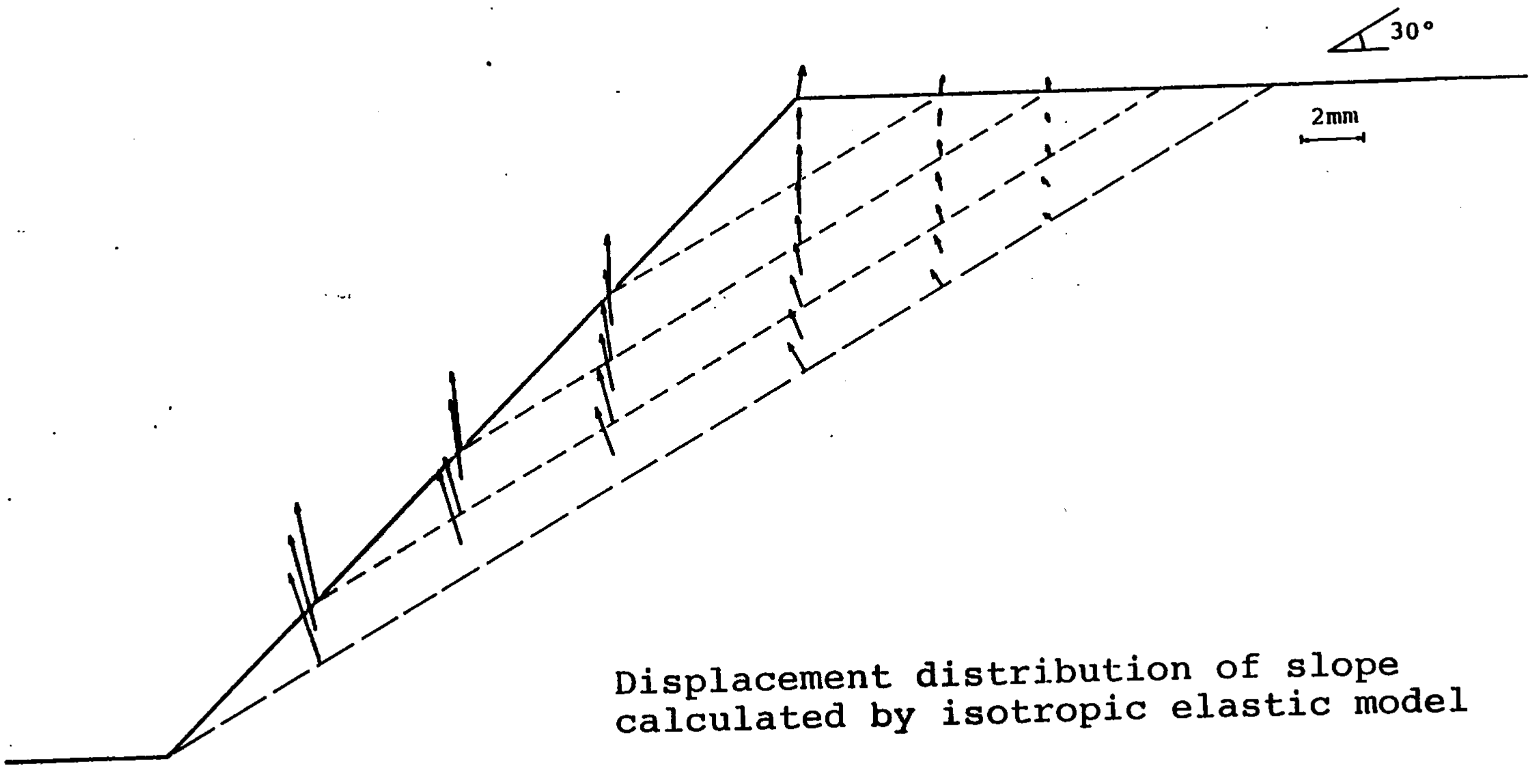
PHYSICAL MODELING OF TOPPLING FAILURE

Laboratory experiment result



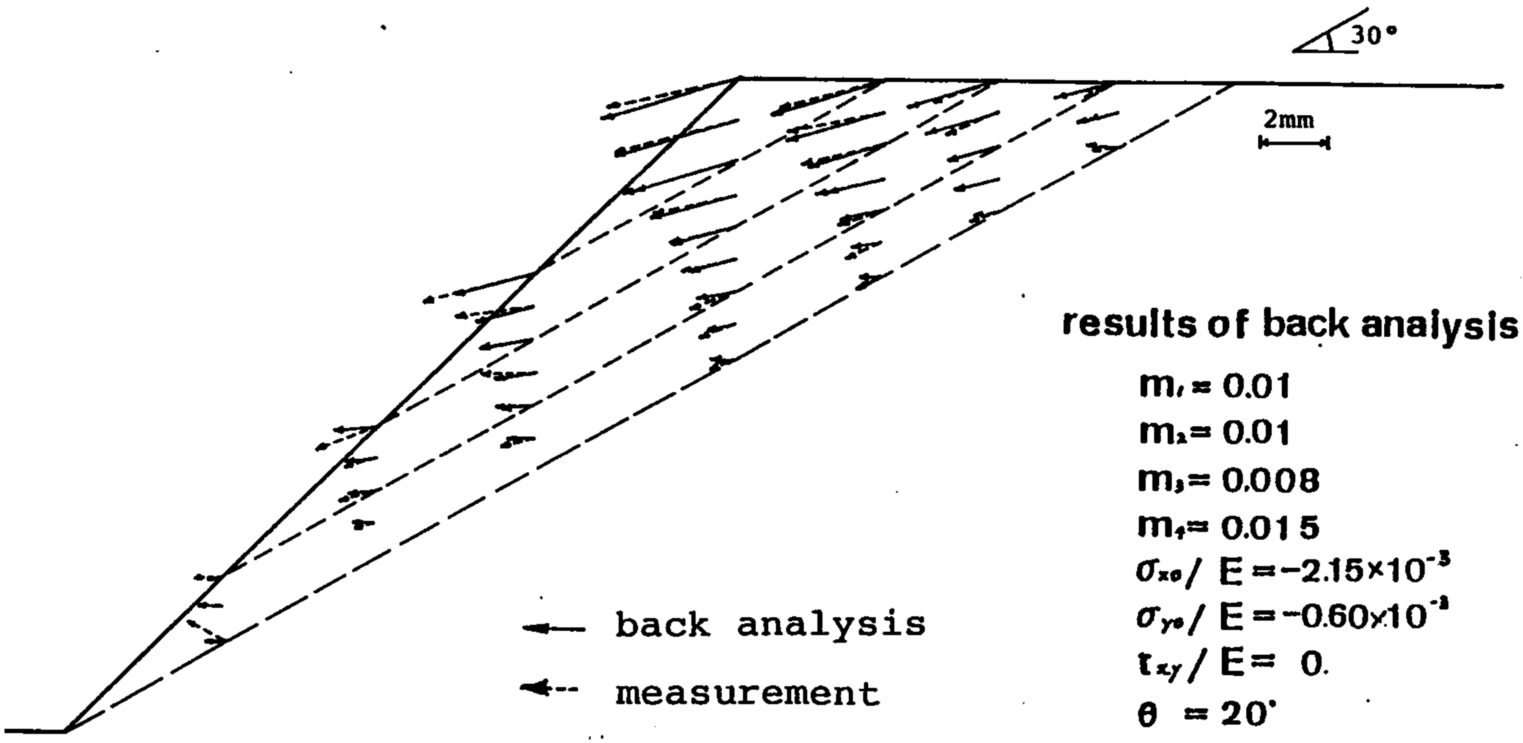
Displacement distribution of slope excavated in jointed materials
(Laboratory test result)

Numerical analysis
(isotropic elastic)



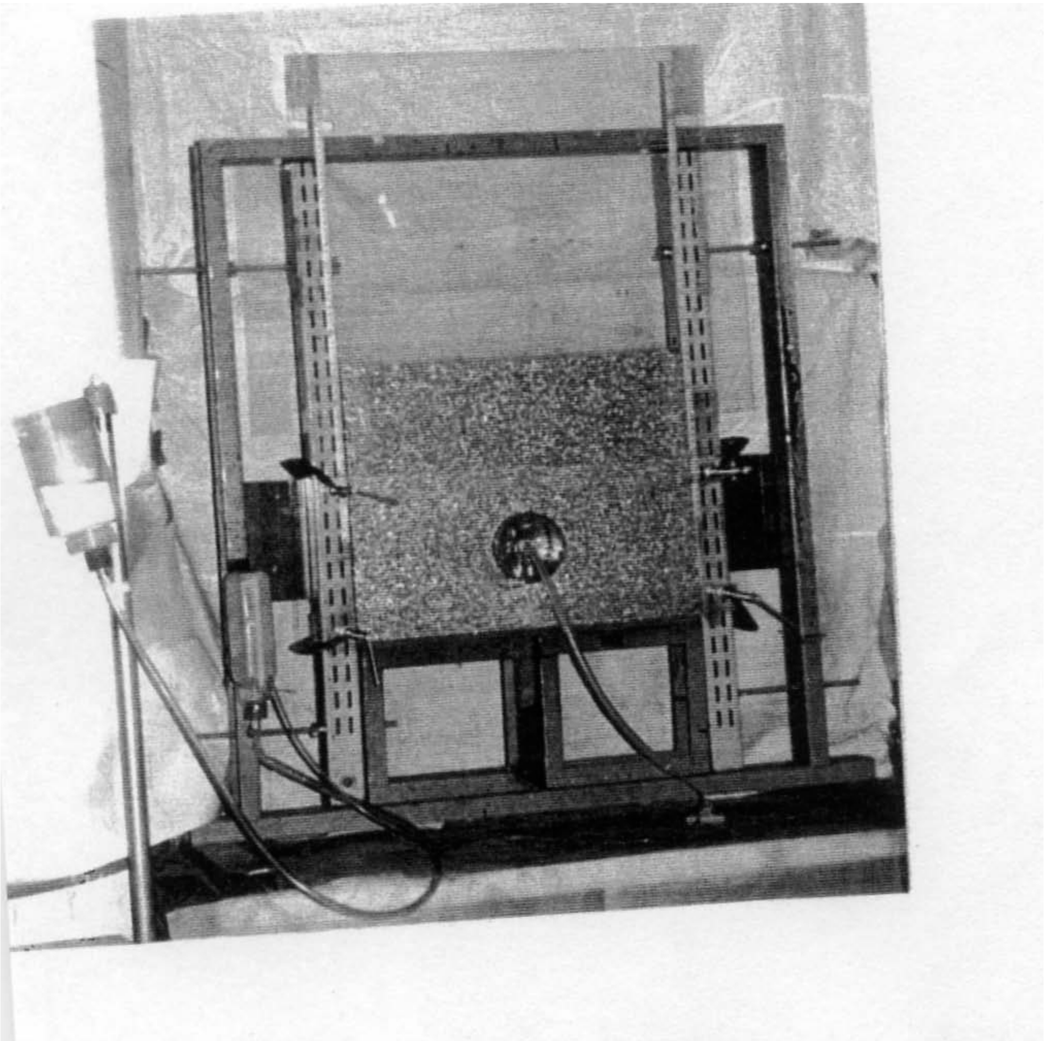
Displacement distribution of slope
calculated by isotropic elastic model

Numerical analysis considering the anisotropic damage parameter

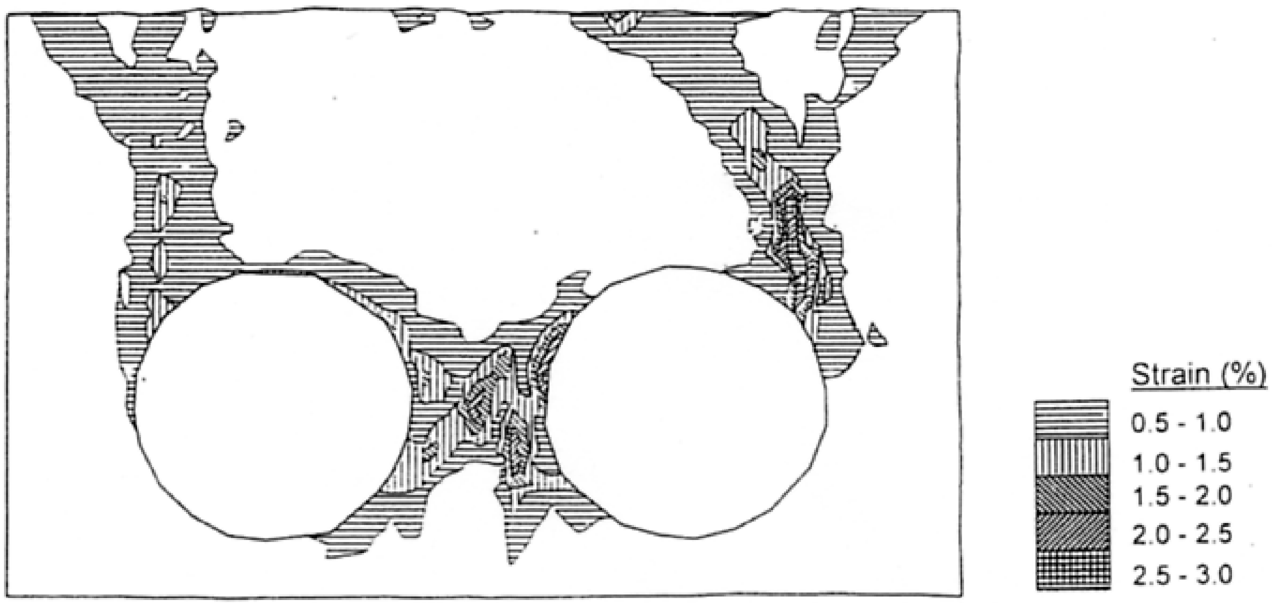


Comparison of measured and back analyzed displacement

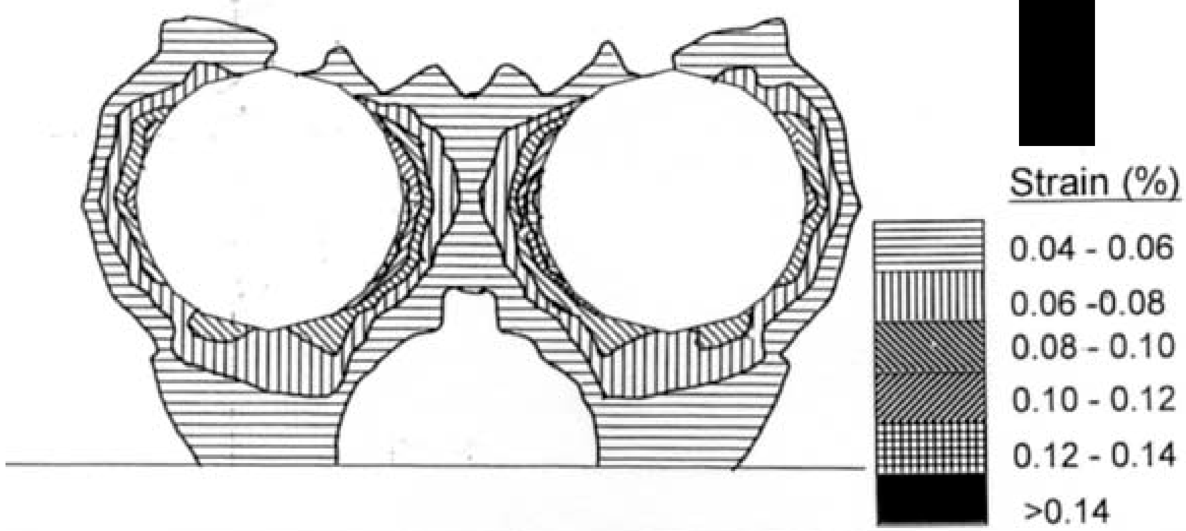
Experimental Device for Simulating Tunnel Excavation (aluminum bars)



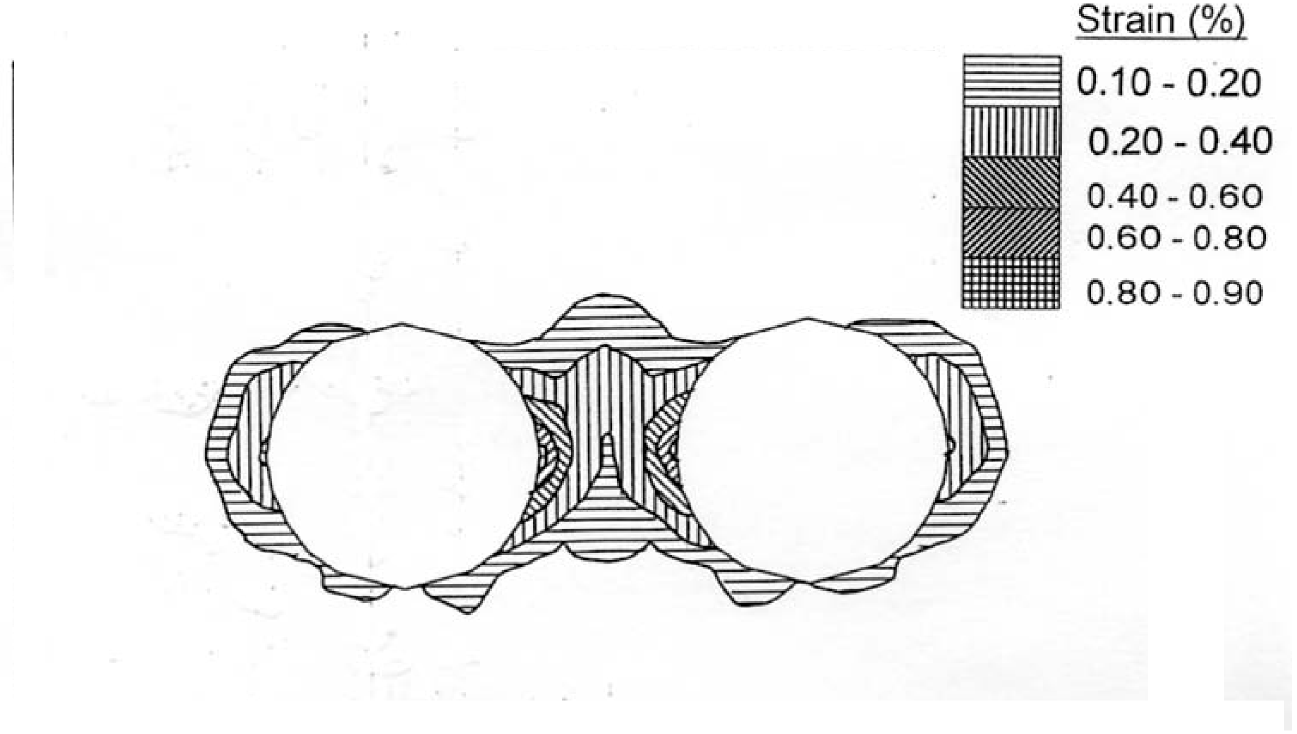
Maximum shear strain distribution around the two parallel tunnels (Experimental results)



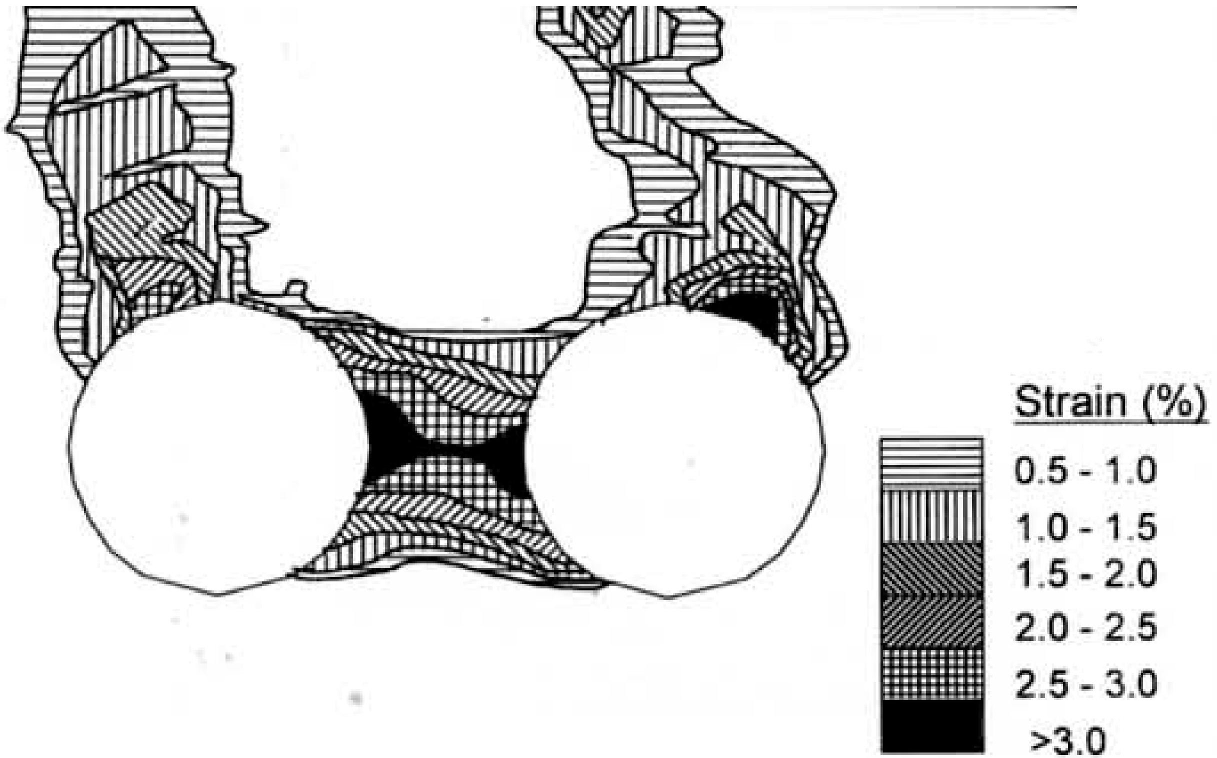
Maximum shear strain distribution (Isotropic elastic analysis)



Maximum shear strain distribution
(Elasto-plastic analysis)



Maximum shear strain distribution
(Taking into account the damage parameter)

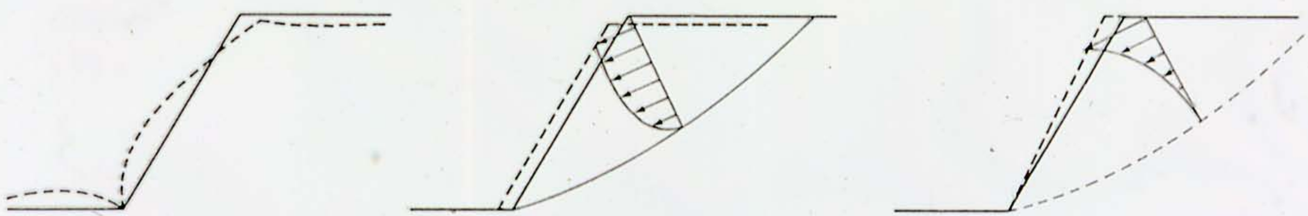


Contents

- Introduction
- Observational method in tunnels (Hazard Warning Level)
- Back analysis
- Anisotropic parameter
- Observational method in slopes
- Case study (Tunnel)
- Case study (Slope)
- Conclusions

Deformational modes of slopes

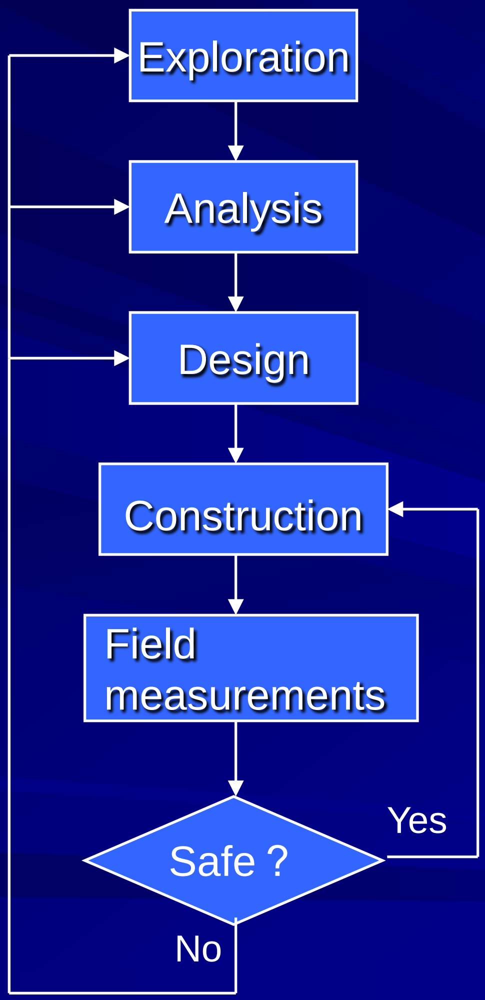
(a) Elastic
(b) Sliding
(c) Toppling

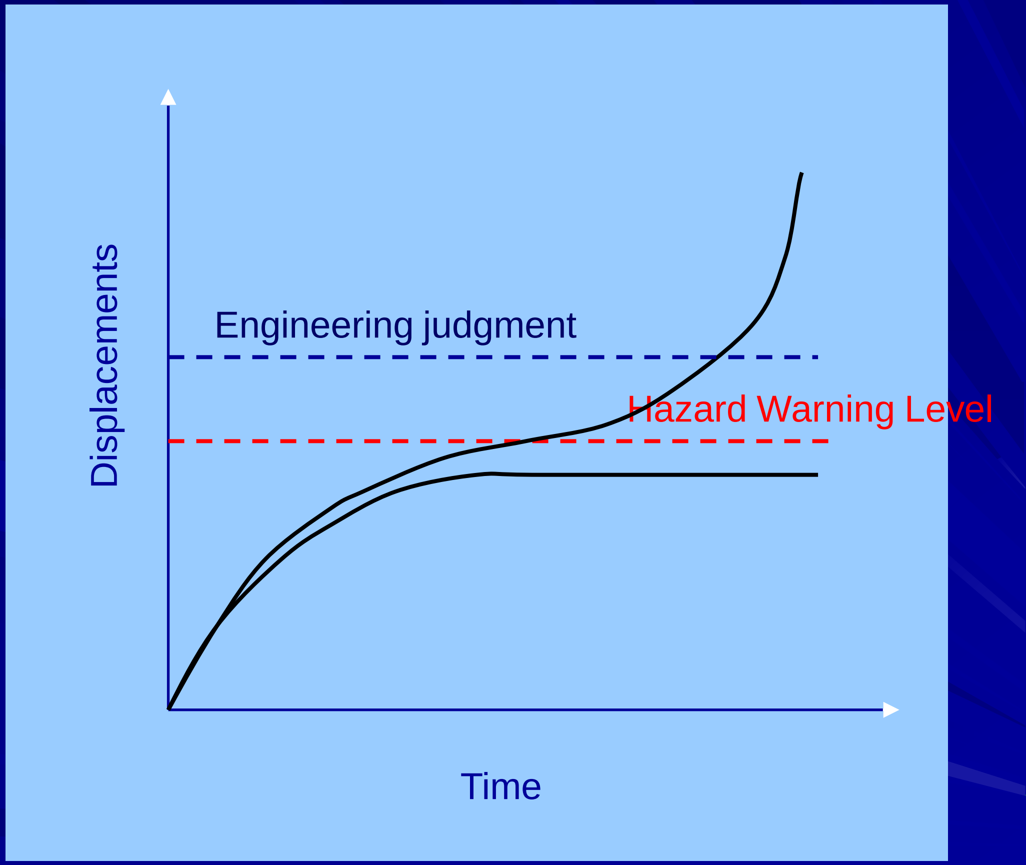


(a) elastic (b) sliding (c) toppling

Deformational modes of cut slopes

Observational method





Interpretation of measured displacements in slopes

- Is it possible to estimate a sliding surface only by surface displacement measurements?

- How can we assess the stability of slopes only by the surface displacement measurements?

- “Back analysis” must be a powerful tool for interpreting the measurement data.

Stress and strain relationship in plane strain condition

$$\{\sigma\} = [\boldsymbol{D}]\{\varepsilon\}$$

Where

$$[\boldsymbol{D}] = \frac{\boldsymbol{E}}{\mathbf{1} - \nu - \mathbf{2}\nu^{\mathbf{2}}} \begin{bmatrix} \mathbf{1} - \nu & \nu & \mathbf{0} \\ \nu & \mathbf{1} - \nu & \mathbf{0} \\ \mathbf{0} & \mathbf{0} & \boldsymbol{m}(\mathbf{1} - \nu - \mathbf{2}\nu^{\mathbf{2}}) \end{bmatrix}$$

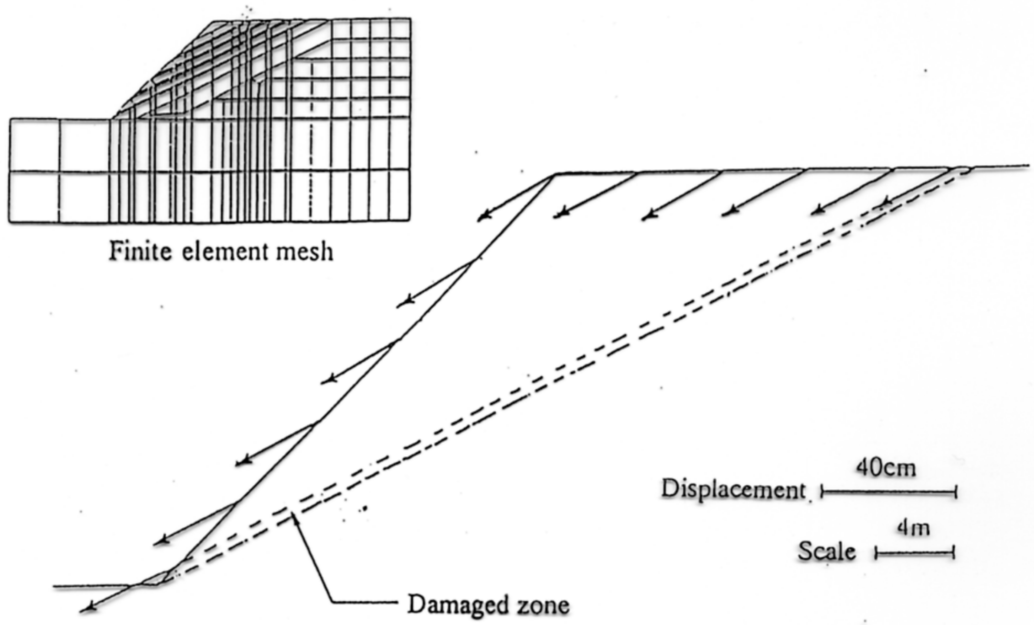
E : Young's modulus

ν : Poisson's ratio

$$\boldsymbol{m} = \mathbf{1} / \mathbf{2}(\mathbf{1} + \nu) - \boldsymbol{d}$$

$\boldsymbol{d}$: Anisotropic damage parameter

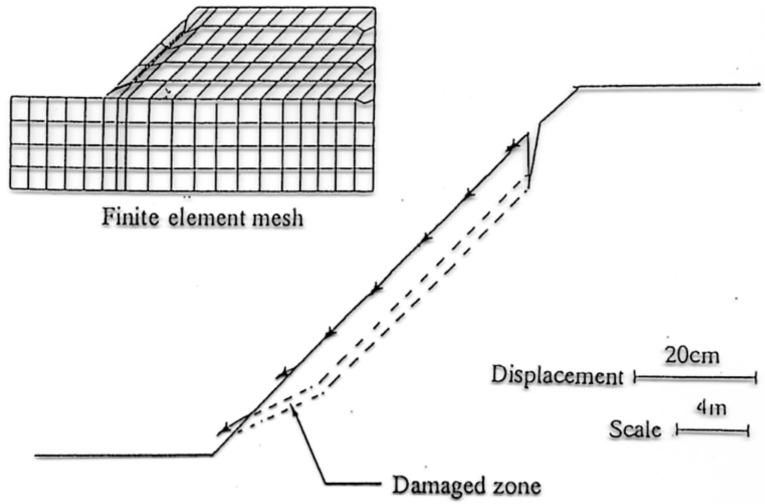
Sliding (straight sliding plane)



Material properties

No.	Young's modulus E (kgf/cm^2)	Unit weight γ (g/cm^3)	Poisson's ratio ν	Anisotropic damage parameters	
				m	α (deg)
1	1024	2.0	0.30	0.385	0.00
2	1024	2.0	0.30	0.001	26.5

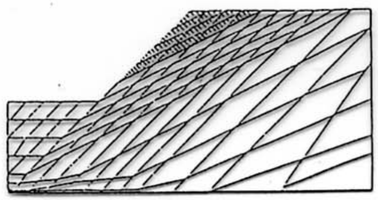
Landslide



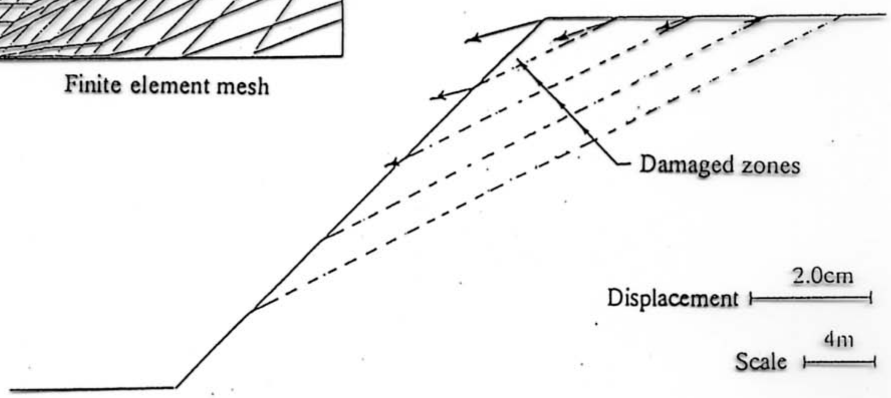
Material properties

No.	Young's modulus E (kgf/cm^2)	Unit weight γ (g/cm^3)	Poisson's ratio ν	Anisotropic damage parameters	
				m	α (deg)
1	1024	2.0	0.30	0.385	0.00
2	1024	2.0	0.30	0.001	24.00
3	1024	2.0	0.30	0.001	45.00
4	1024	2.0	0.30	0.001	45.00
5	1024	2.0	0.30	0.001	45.00

Toppling



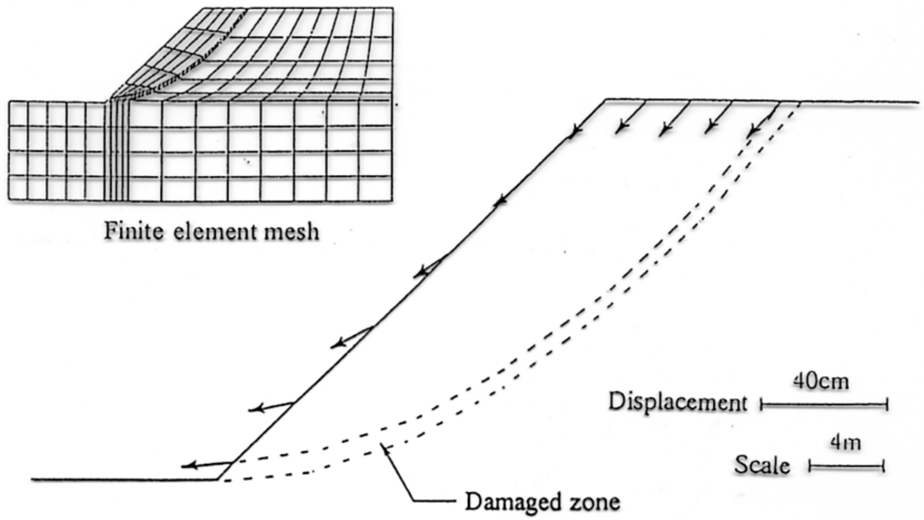
Finite element mesh



Material properties

No.	Young's modulus E (kgf/cm²)	Unit weight γ (g/cm³)	Poisson's ratio ν	Anisotropic damage parameters	
				m	α (deg)
1	1024	2.0	0.30	0.385	0.00
2	1024	2.0	0.30	0.005	15.00
3	1024	2.0	0.30	0.01	15.00
4	1024	2.0	0.30	0.05	15.00
5	1024	2.0	0.30	0.1	15.00

Sliding (curved sliding plane)

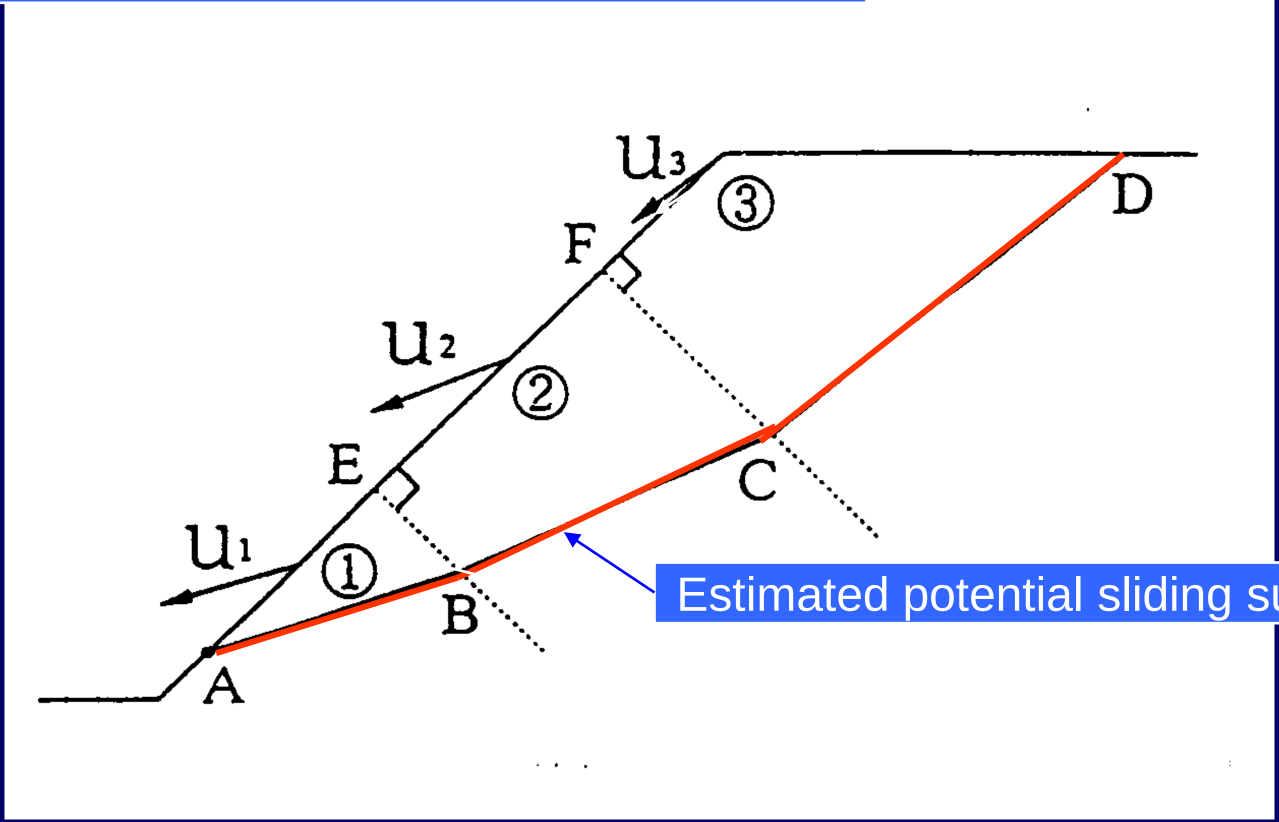


Material properties

No.	Young's modulus E (kgf/cm^2)	Unit weight γ (g/cm^3)	Poisson's ratio ν	Anisotropic damage parameters	
				m	α (deg)
1	1024	2.0	0.30	0.385	0.00
2	1024	2.0	0.30	0.001	6.39
3	1024	2.0	0.30	0.001	18.37
4	1024	2.0	0.30	0.001	29.07
5	1024	2.0	0.30	0.001	37.88
6	1024	2.0	0.30	0.001	45.00
7	1024	2.0	0.30	0.001	50.71

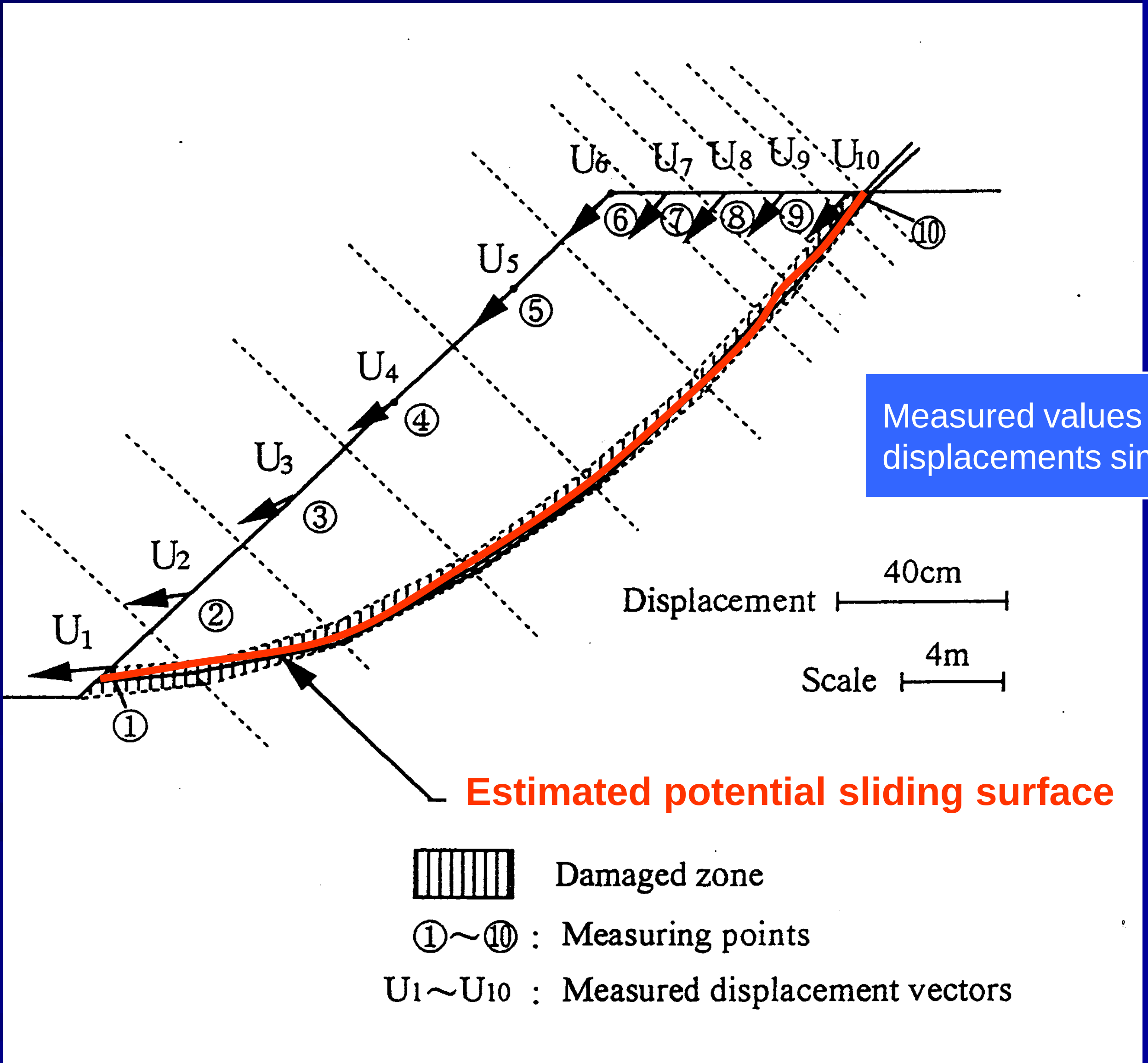
Estimation of a sliding surface from surface displacements

u_1, u_2, u_3 : Measured displacements



Estimated potential sliding surface

Numerical simulation for demonstrating accuracy of the proposed method



Measured values of surface displacements simulated by FEM

Design concept of slope

- Factor of safety:

$$F = \frac{\Sigma R_i}{\Sigma T_i} \geq 1.0$$

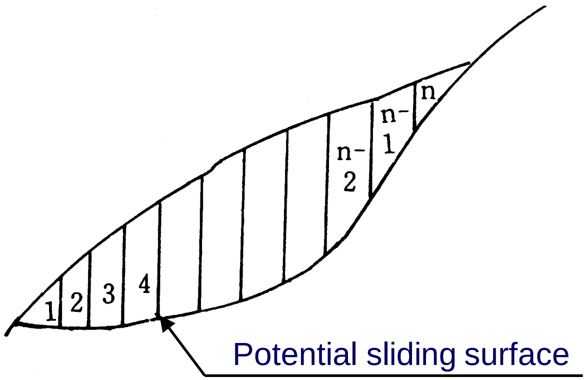
where ΣR_i : shear strength

ΣT_i : sliding force

$$R_i = f(c, \phi \ldots\ldots)$$

- We must estimate strength parameters (c, φ) .

Limit Equilibrium Analysis



$T_i = W_i \cdot \sin\theta_i$

$R_i = (W_i \cdot \cos\theta_i - U_i)\tan\phi + c l_i$

W_i ：分割片の重量（t／m）

θ_i ：すべり面の分割片部における傾斜角(度)

U_i ：分割片に働く間隙水圧（t／m）

l_i ：分割片のすべり面延長（m）

Design vs. Monitoring in Slope

- In the design of slopes, Young's modulus is no need. Only strength parameters (C, φ) are used.

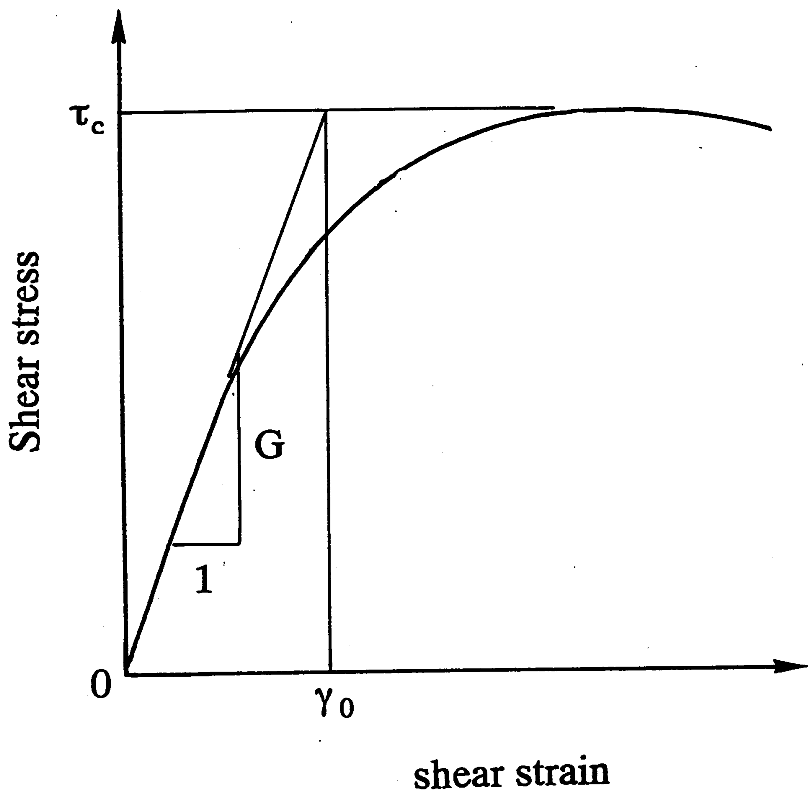
- However, in monitoring the stability of slopes displacement measurements such as extensometer, inclinometer, total station, GPS, etc. are commonly used.

Question is how to assess the measured displacements.

How can we determine
strength parameters (c, φ) of soils/rocks
from measured surface displacements?

Answering this question,
we proposed “Critical Shear Strain”.

Definition of critical shear strain



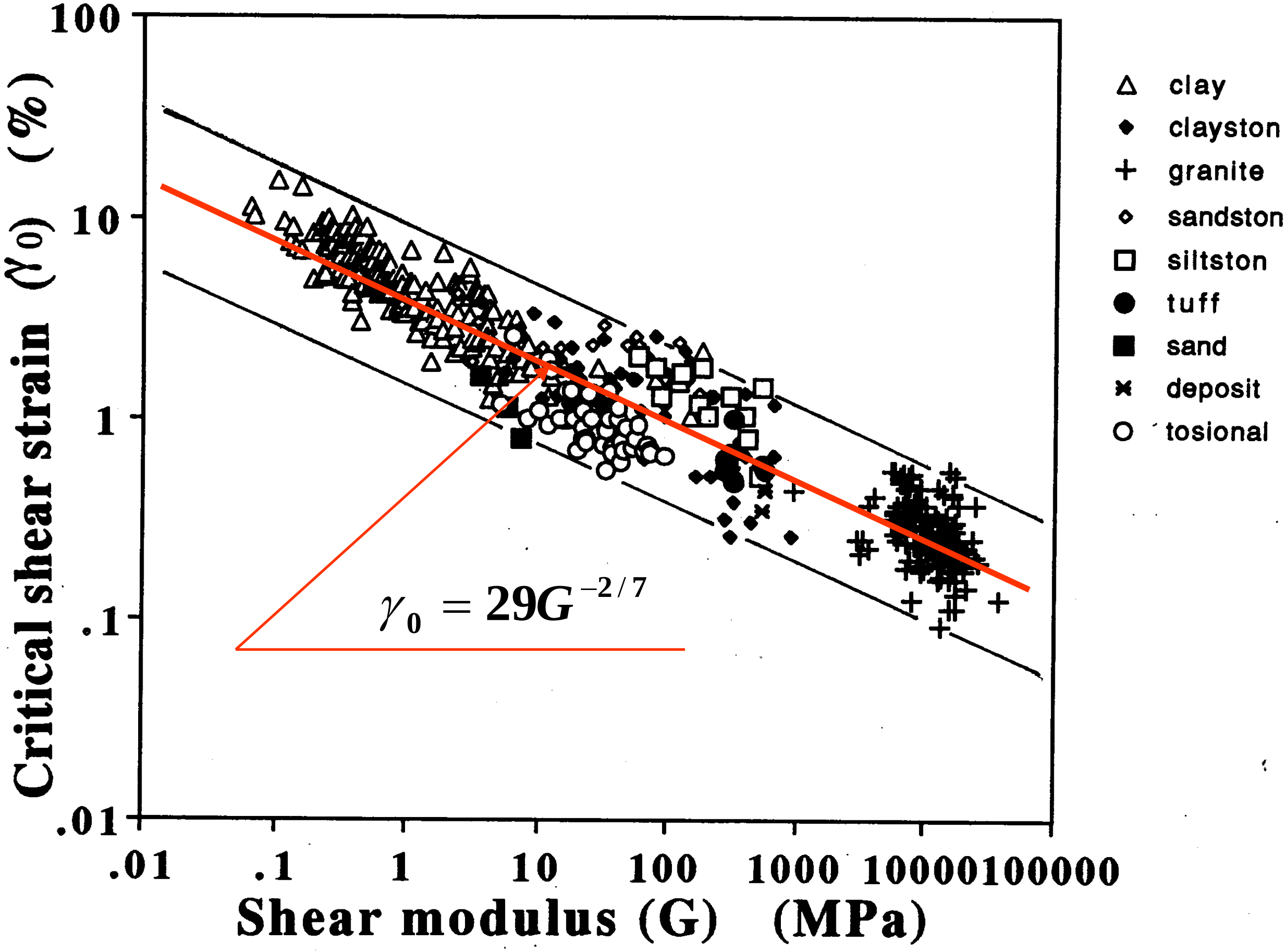
Critical shear strain γ_0

$\gamma_0 = \tau_c / G$

where τ_c : ***shear strength***

G : ***shear modulus***

Laboratory experimental results
(Relationship between critical shear strain and shear modulus)



Procedure of determining the strength parameters from measured displacements

Displacement measurements (GPS)

↓

Determination of Young's Modulus E and anisotropic parameter *m* by back analysis of measured displacements

$$\Delta u^2 = \sum_{i=1}^{M} (u_i^c - u_i^m)^2 \rightarrow \min$$

↓

Shear Modulus G = *m*E

↓

Critical shear strain $\gamma_0 = f(G)$

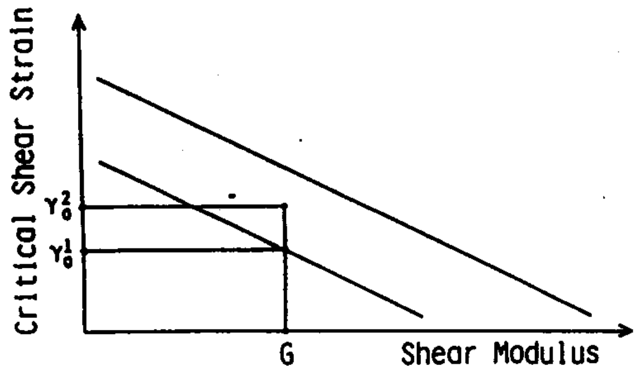
↓

Shear strength $\tau_c = G\gamma_0$

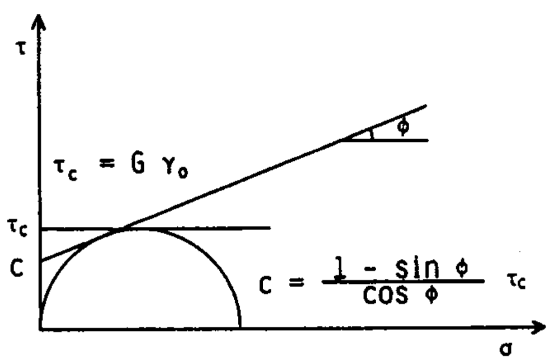
↓

Strength parameter (Cohesion) $c = \dfrac{1-\sin\phi}{\cos\phi}\tau_c$ (φ: assumed)

Procedure of determining the strength parameters (c, φ)



(a) Relationship between shear modulus and critical shear strain



(b) Shear strength

Schematic diagram for critical shear strain and shear strength

- Young's Modulus E and the parameter *m* are obtained by back analysis.

- Shear Modulus G is determined.

  G = *m*E

- Critical shear strain γ_0 is then obtained by the figure (a).

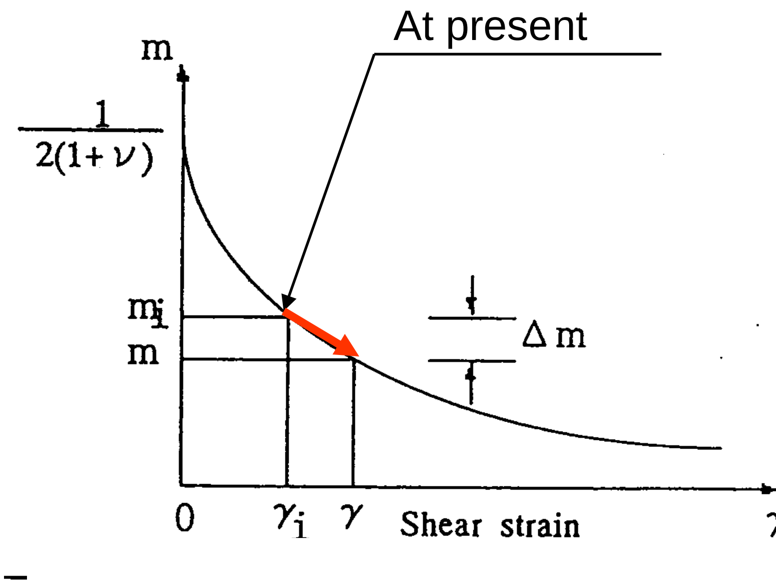
- Then, the shear strength τ_c , and c and **φ** can be determined by,

$$\tau_c = G\gamma_0$$

$$c = \frac{1-\sin\phi}{\cos\phi}\tau_c$$

FINITE ELEMENT ANALYSIS

The relationship between the increment of external forces and increment of displacements is expressed as,



$$[K]\{\Delta\delta\} = \{\Delta P\}$$

where

$[K]$: Stiffness matrix

$\{\Delta\delta\}$: Increment of displacements Some of them are measured values.

$$\{\Delta P\} = \int [B]^T [D][\Delta C]\{\sigma\} dV \quad + \int [B]^T \{\sigma\} dV$$

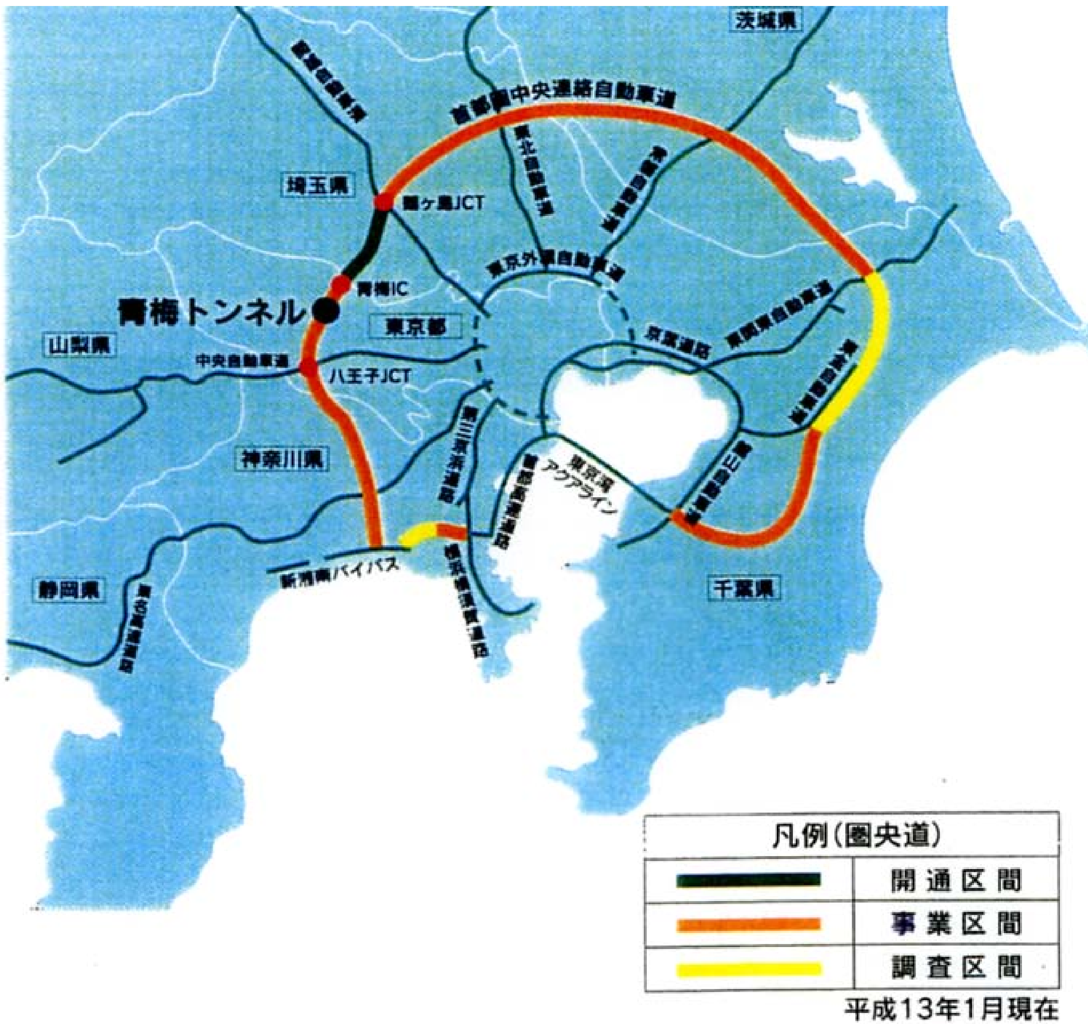
where

$$[\Delta C] = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & \dfrac{\Delta m}{m \quad m_i} \end{bmatrix}$$

Contents

- Introduction
- Observational method in tunnels (Hazard Warning Level)
- Back analysis
- Damage parameter
- Observational method in slopes
- Case study (Tunnel)
- Case study (Slope)
- Conclusions

Case study (Ohme Tunnel)



The" Ohme Tunnel" is located in a highly populated residential area. Limitation of the tunnel width is 16-20m.

Length : 2,095 m
(NATM: 1,093 m, Cut-and-cover: 1,002m)

Depth: 8.5 m (average)

Geology: Diluvial formation

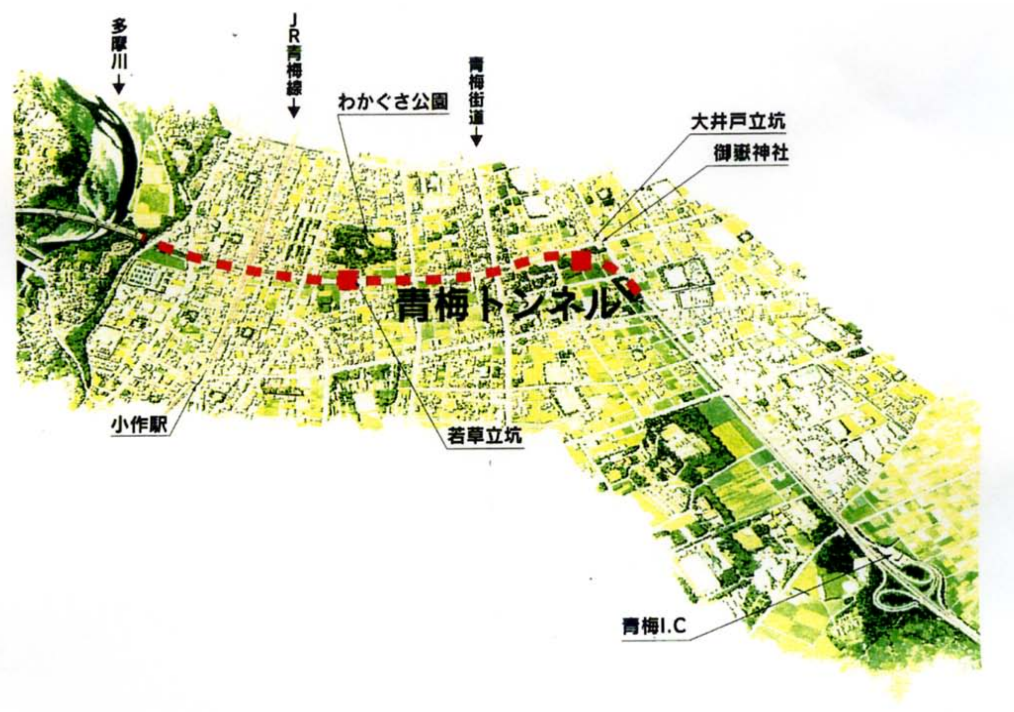
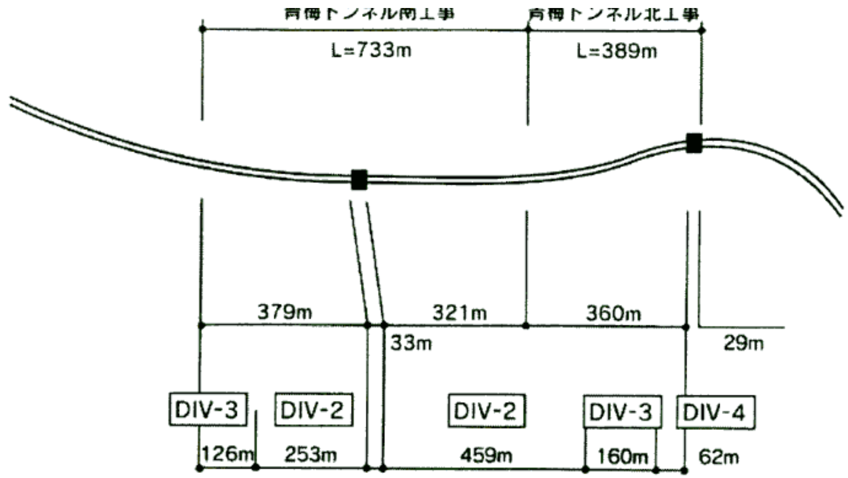


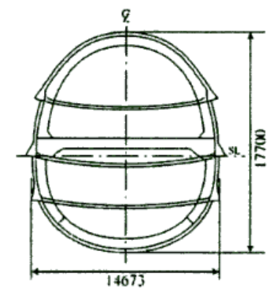
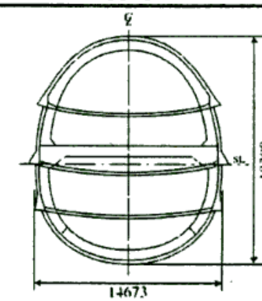
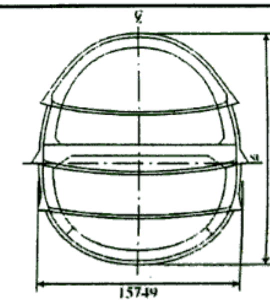
図 1.1.2 青梅トンネル周辺位置図

Longitudinal cross section
Double deck

図 1.1.3 全体計画図（縦断図）

Cross section



断面区分		DIV-2	DIV-3	DIV-4
断面図				
吹付コンクリート厚さ(mm)		250	250	250
覆工厚さ(mm)		800	800	800
中床版厚さ(mm)		900	900	1000
掘削(m²/m)	1段	63.1	63.1	67.7
	2段	65.8	65.8	70.6
	3段	48.7	49.1	52.9
	4段	36.8	44.3	49.0
	合計	214.4	222.3	240.2
吹付コンクリート厚さ(m²/m)		77.7	78.8	82.7
覆工コンクリート(m²/m)	中床版	12.7	12.7	15.1
	上部覆工	18.8	18.8	19.3
	インバート	8.7	8.5	9.7
	下部覆工	11.4	12.5	12.2
	合計	51.6	52.5	56.3
摘要				視距拡幅断面

Support system during excavation

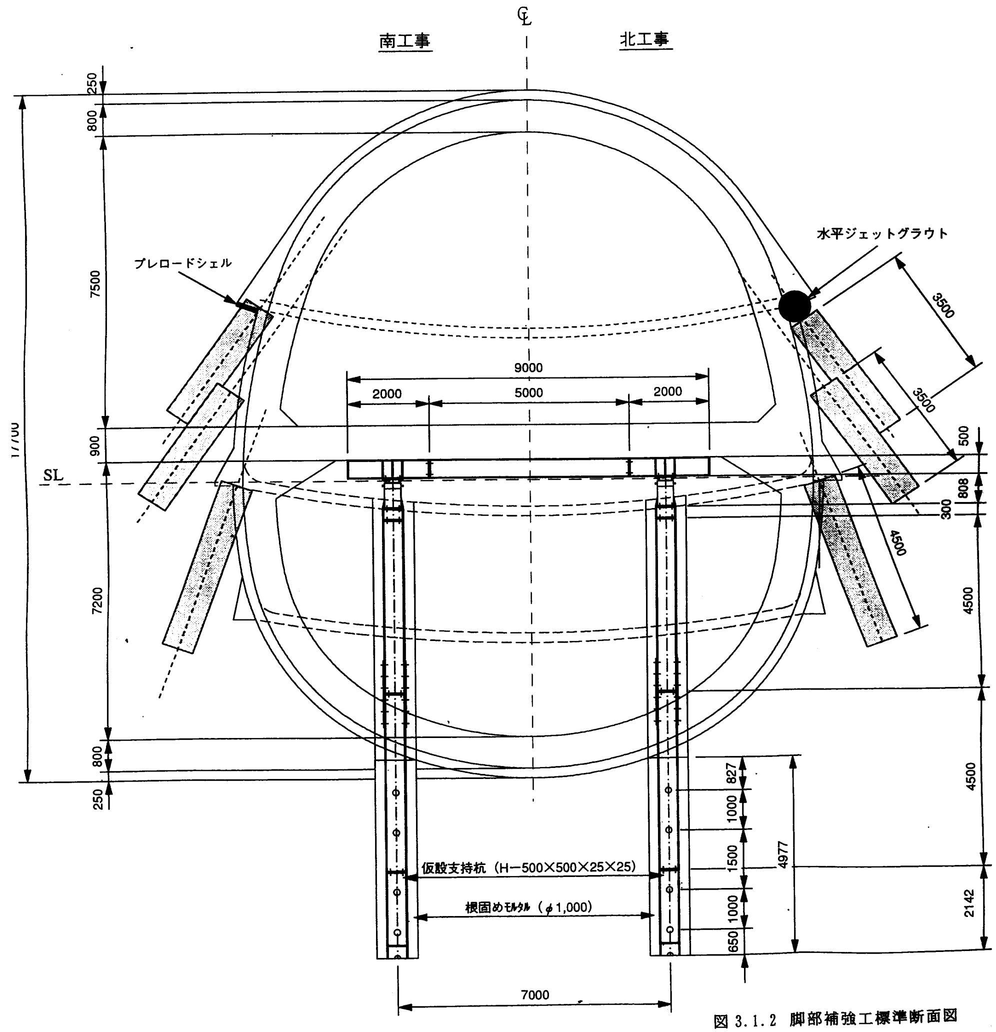
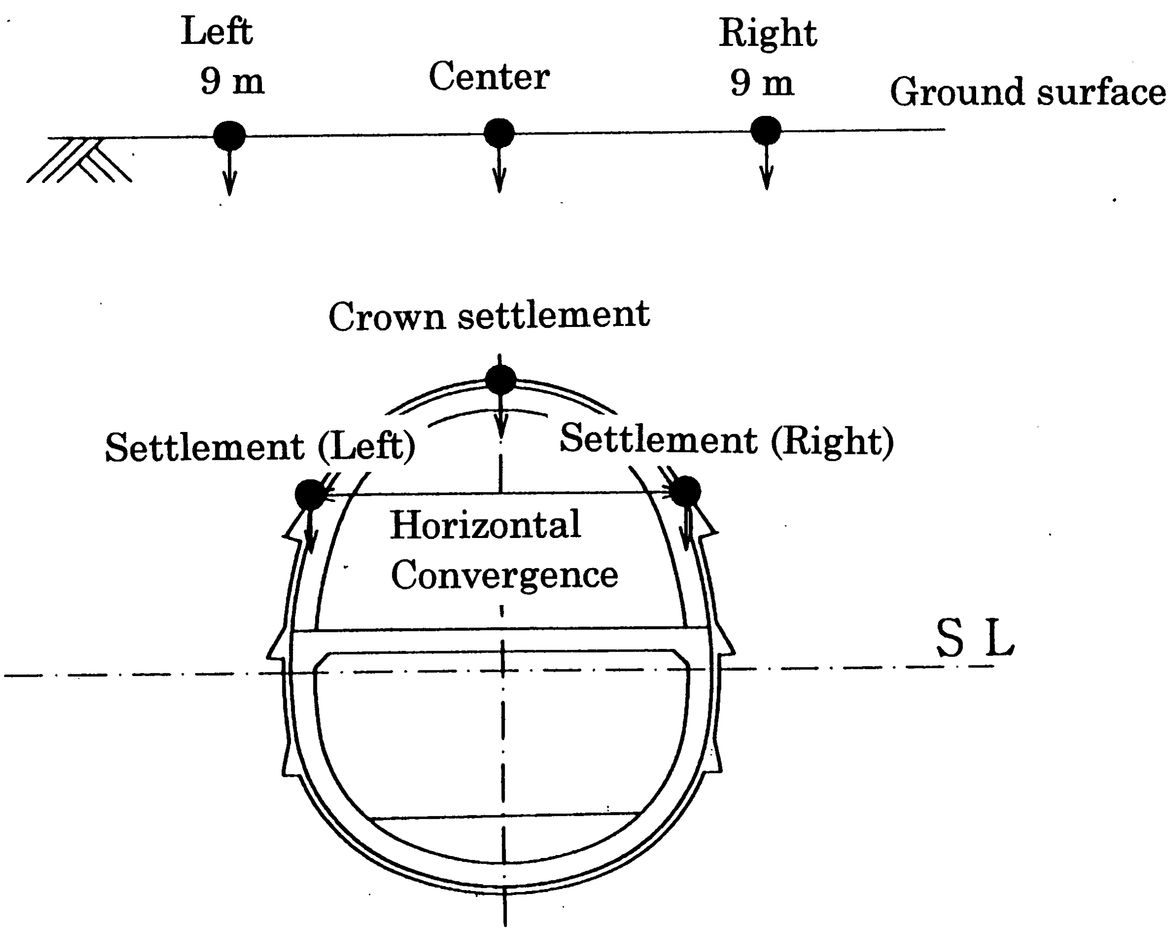


図 3.1.2 脚部補強工標準断面図

Location of measuring points



Comparison between the results of back analyses

Elastic analysis and damage analysis considering the anisotropic damage parameter

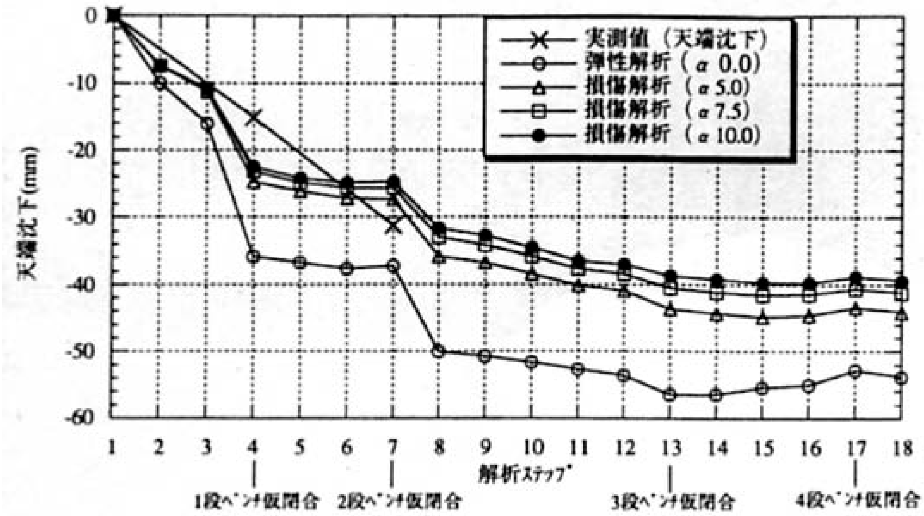


図　天端沈下の経時変化図

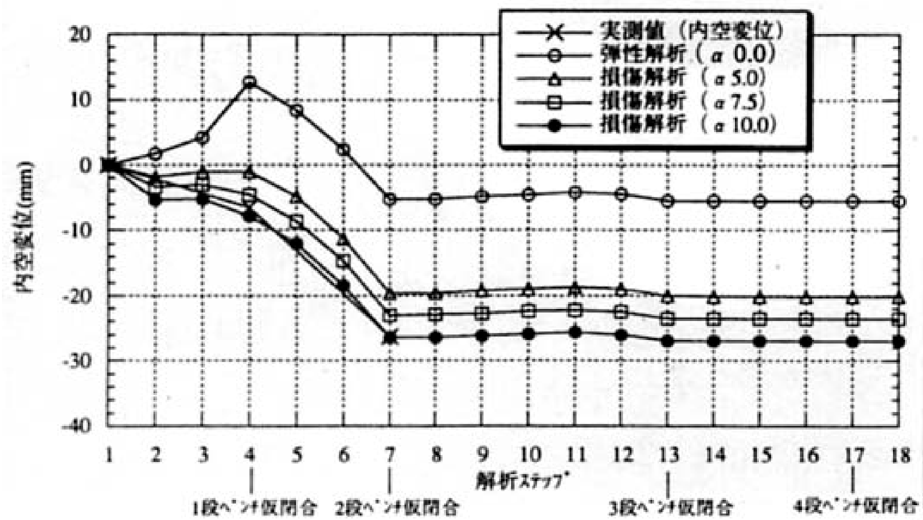


図　内空変位の経時変化図

Measurement results
Horizontal displacements

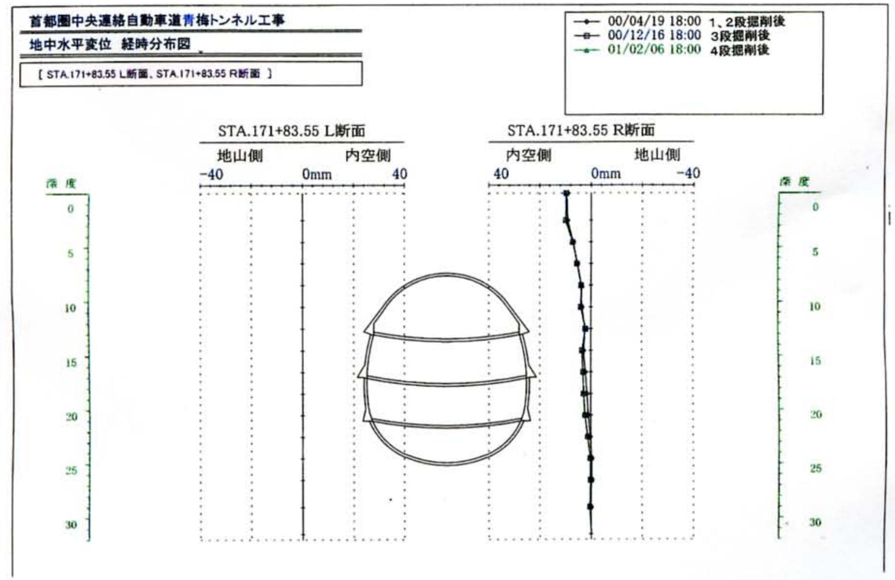


図 3.2.10 地中水平変位経時変化図（南工事：位置図中①）

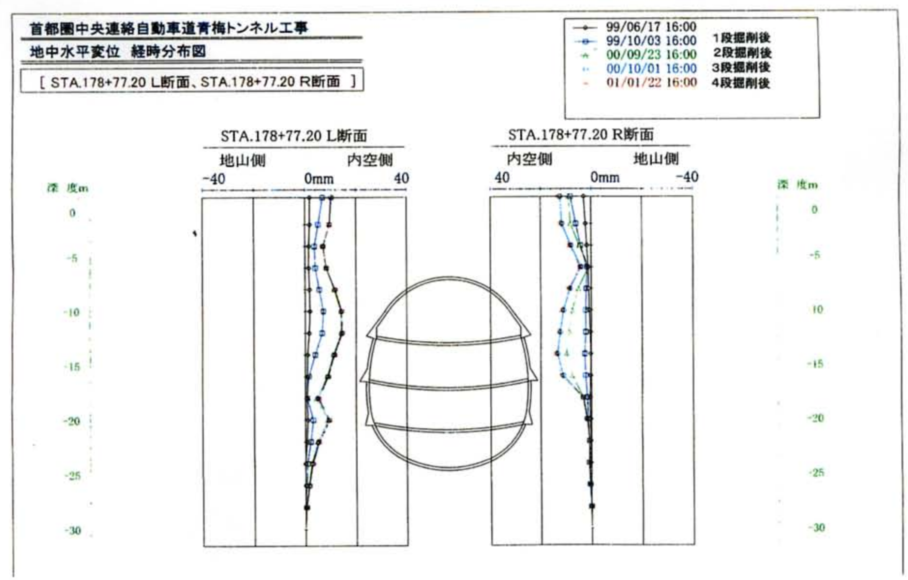
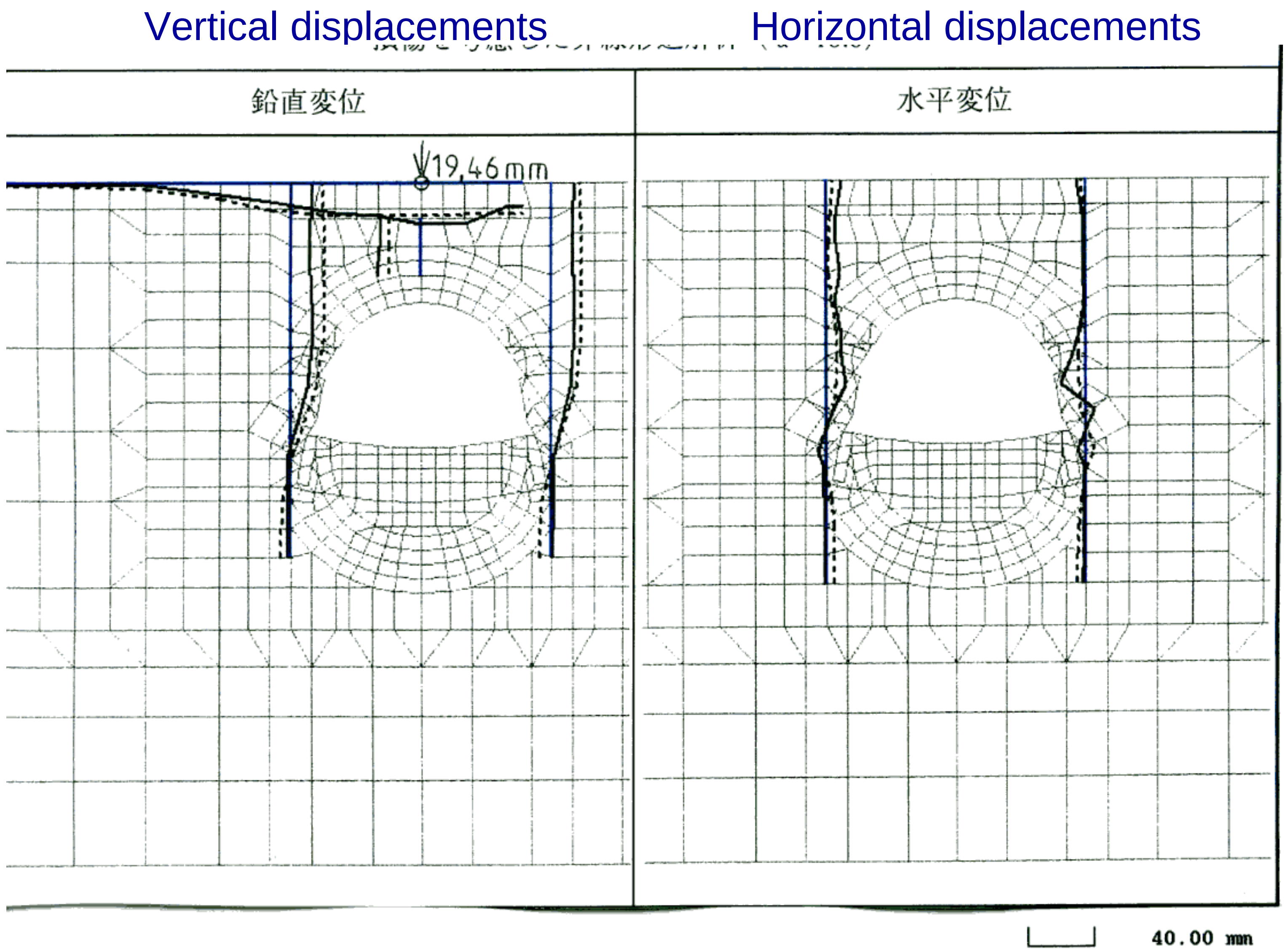


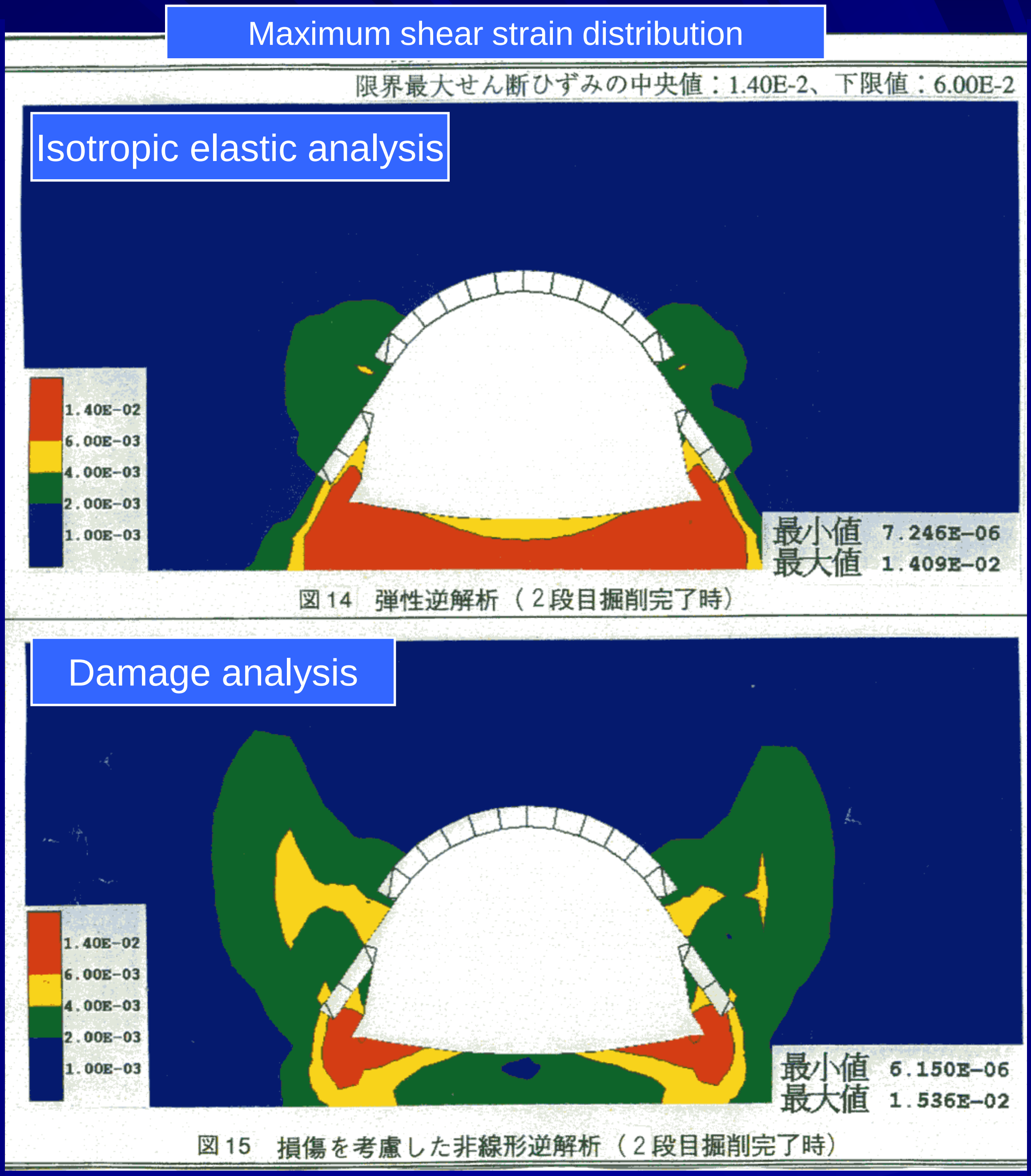
図 3.2.11 地中水平変位経時変化図（北工事：位置図中②）

Comparison between the results of back analyses
Elastic analysis and damage analysis considering the anisotropic damage parameter



Maximum shear strain distribution obtained by back analysis

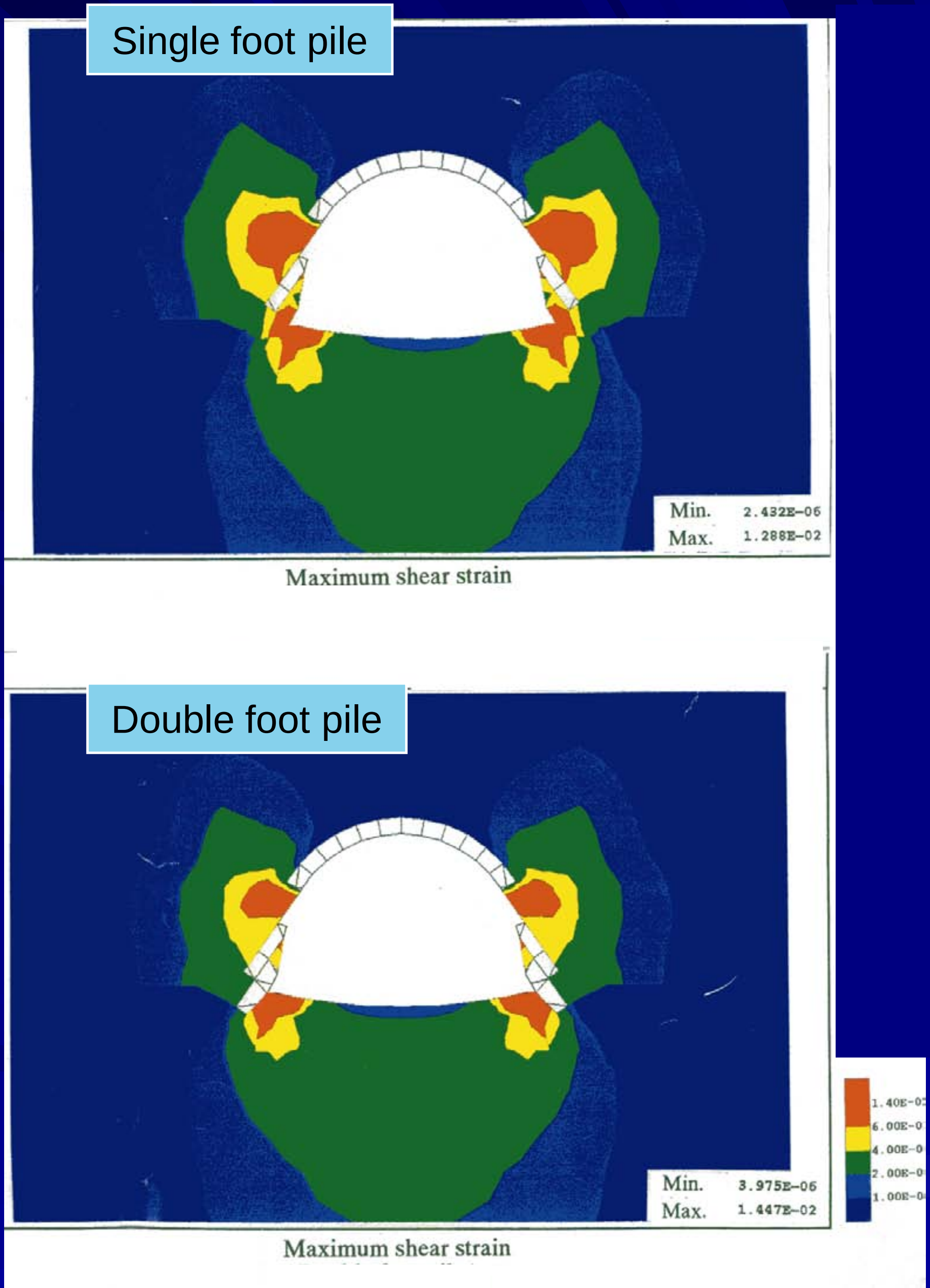
(after Iwano)



Maximum shear strain distribution predicted by using back analysis results

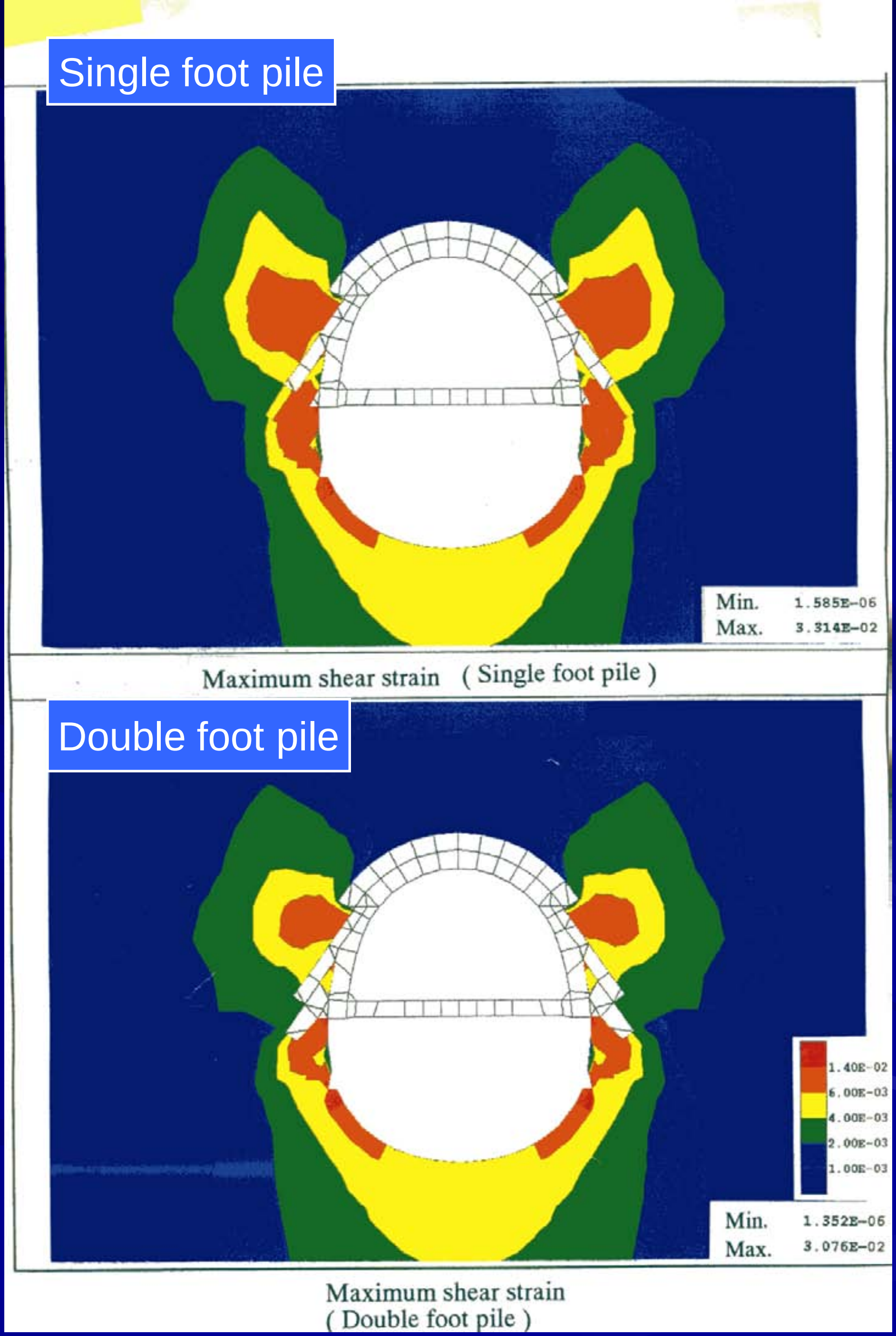
（Comparison between single foot pile and double foot pile ）

（after Iwano)



Maximum shear strain distribution predicted by back analysis results

(after Iwano)



Stability assessment of tunnels

The maximum shear strain should be smaller than the critical shear strain.

$$\gamma_{\max} < \gamma_0 \quad \text{------------} \quad \textit{Stable}$$

If the maximum shear strain becomes larger than the critical shear strain, tunnel support measures must be installed so as to keep the maximum shear strain remain being less than the critical shear strain.

$$\gamma_{\max} > \gamma_0 \quad \text{-----------} \quad \textit{Unstable}$$

Contents

- Introduction
- Observational method in tunnels (Hazard Warning Level)
- Back analysis
- Damage parameter
- Observational method in slopes
- Case study (Tunnel)
- Case study (Slope)
- Conclusions

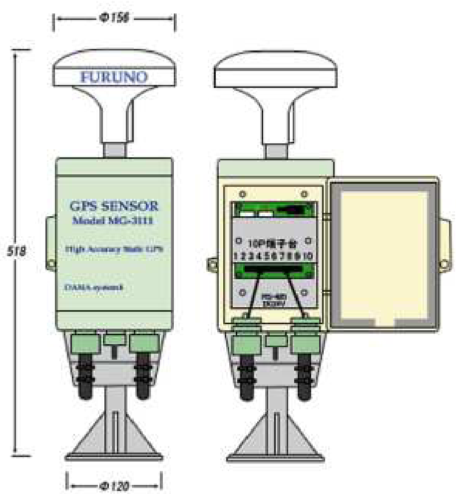
shamen-net

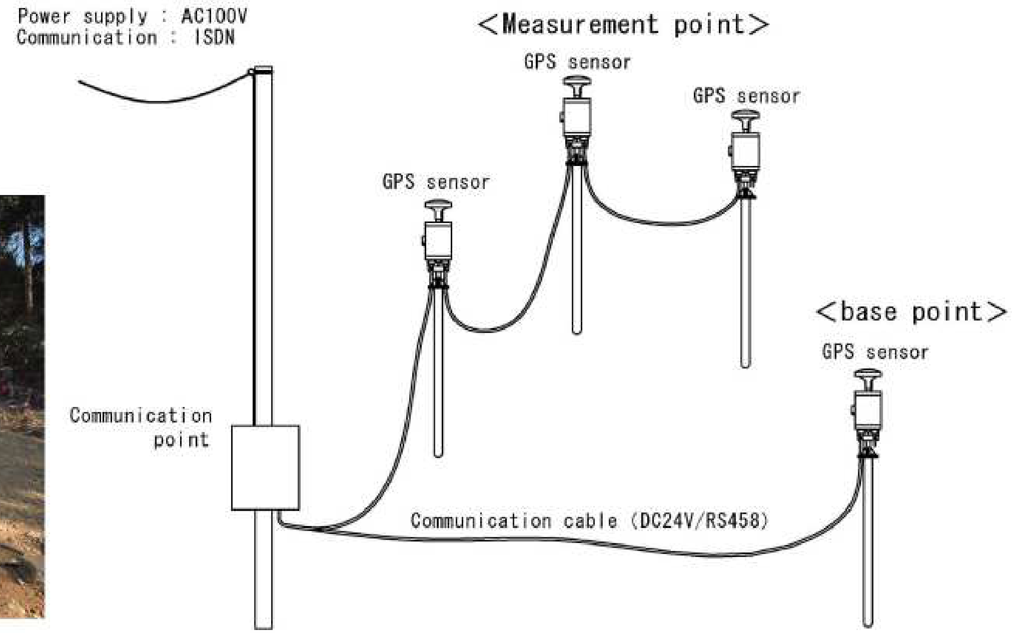
Web-based
GPS Automatic Monitoring Service

cut-slope (expressway)

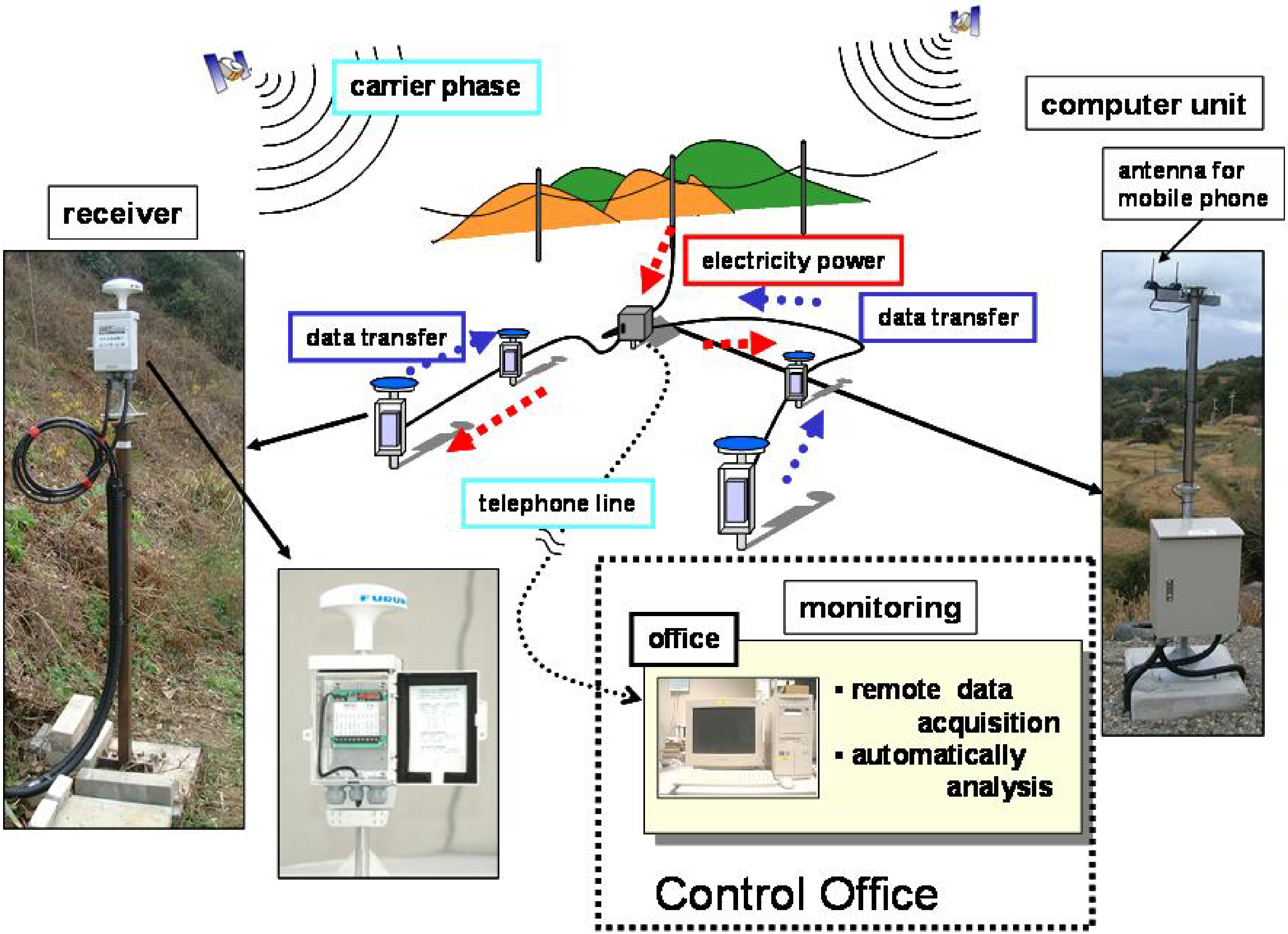
The outline of the GPS machine

GPS : static positioning , L1-band
Measurement interval : 1 hour
Power supply : AC100V
Communication : ISDN (or ADSL or LAN)



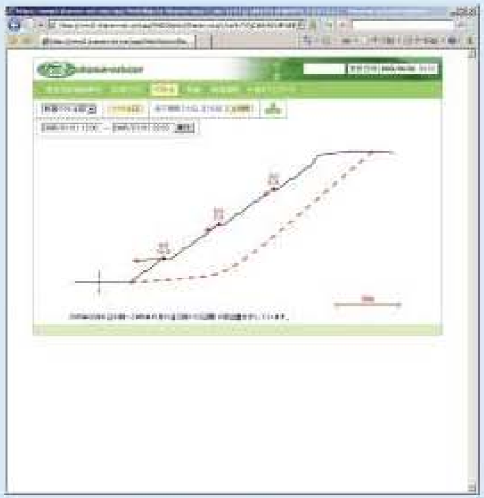


GPS Displacement Monitoring System

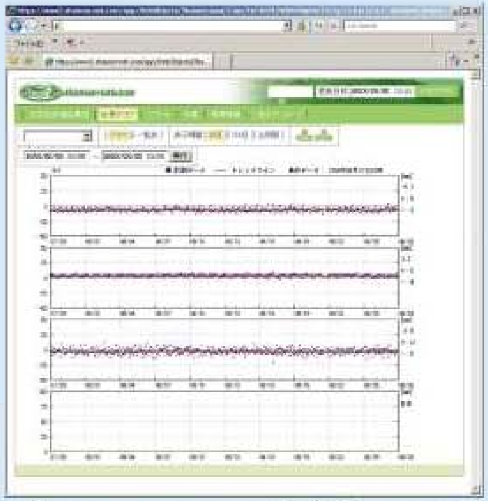


The results are provided on the Internet

Displacement vector (horizontal)

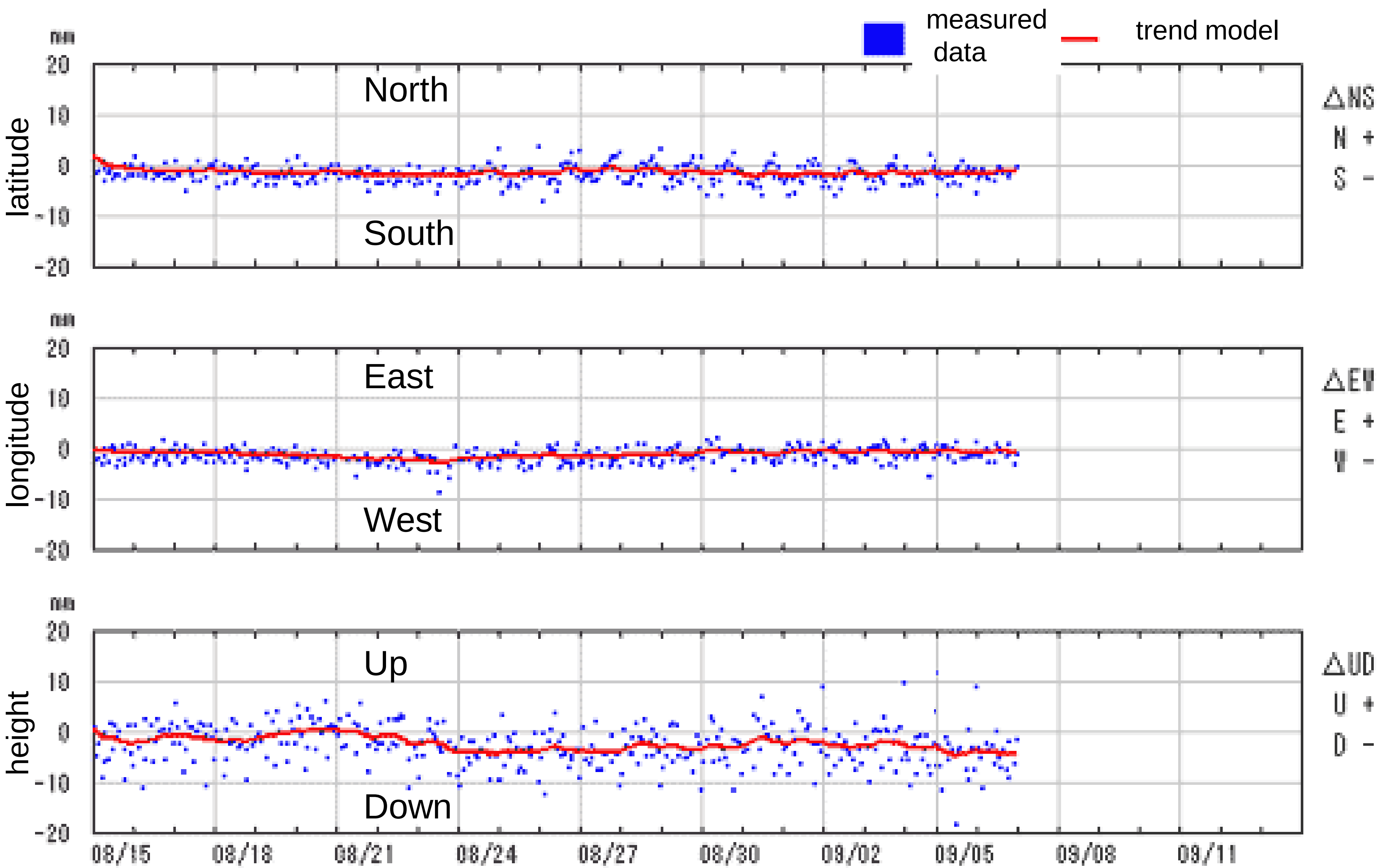


Displacement vector (cross section)



Displacement graph (time series)

Measurement results



Accuracy of the system

The system can detect

displacements : 1~2 mm

displacement velocities : 0.1 mm/day

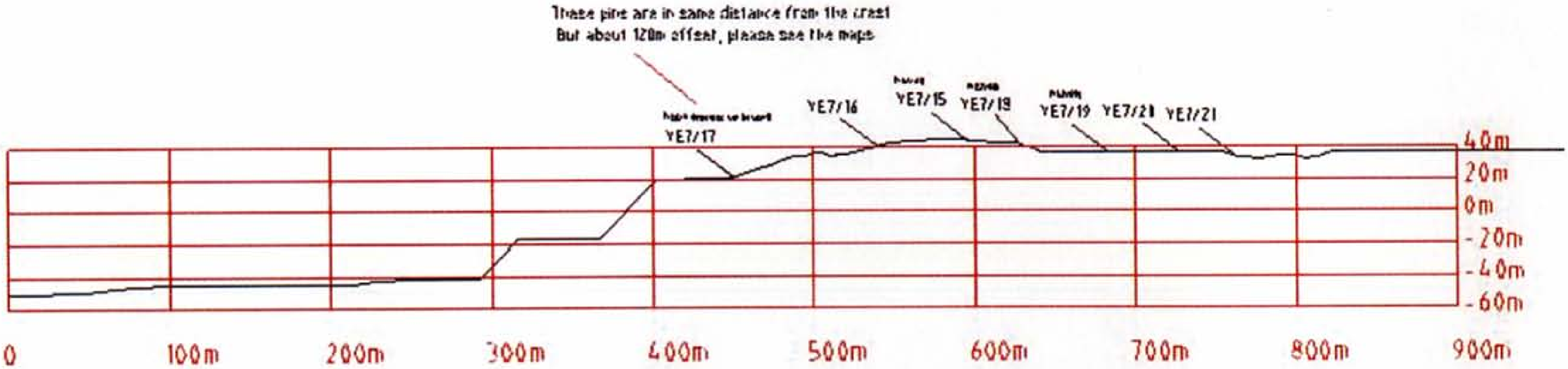
Case study (Open pit coal mine)

In order to verify the applicability of the back analysis method for assessing the stability of slopes, the method was applied to an open pit coal mine, where displacements were measured by GPS until failure occurred.

The strength parameters such as cohesion and internal friction angle of the materials were determined from the measured displacements, and the factor of safety was calculated just before the failure occurred.

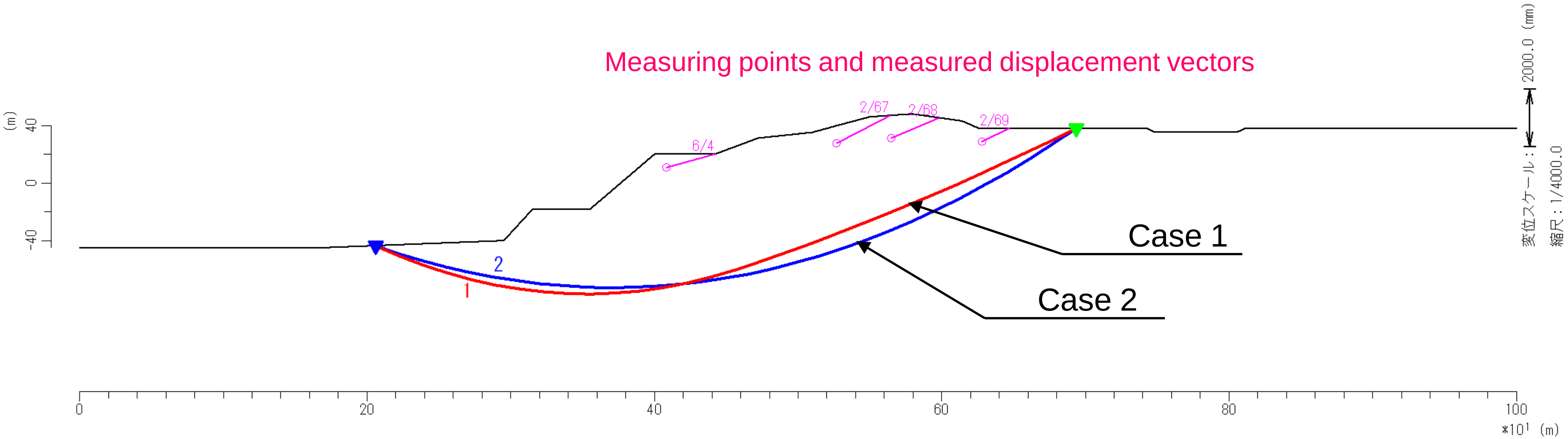
Cross section of the open pit coal mine

Measuring points of GPS are shown on the ground surface.

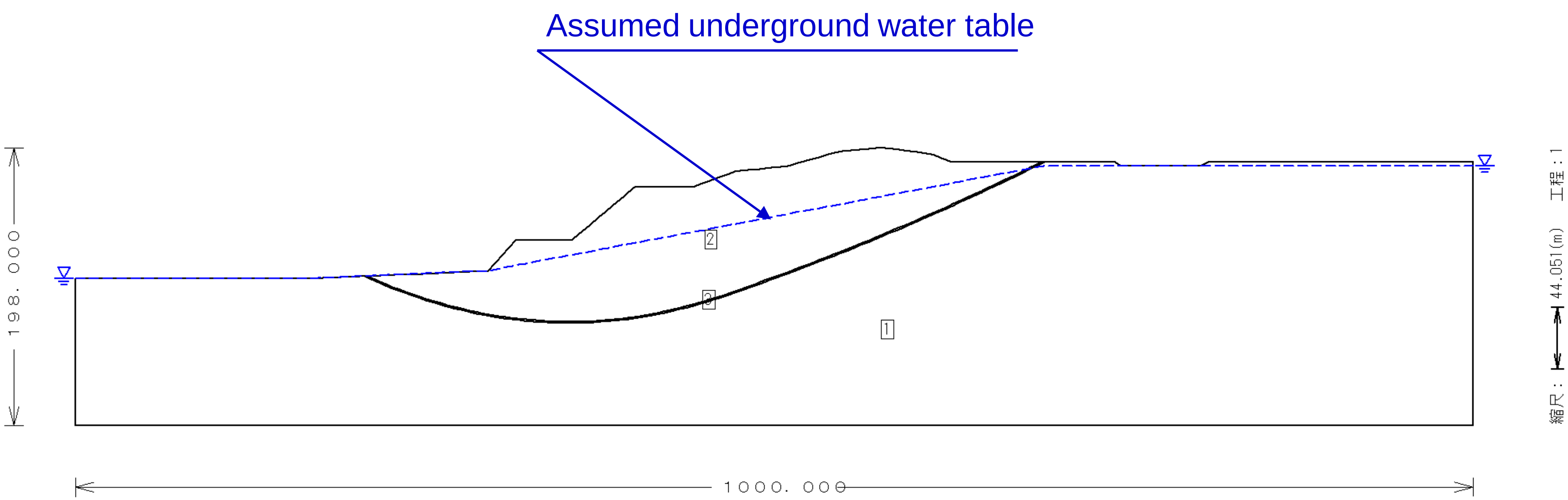


Sliding planes estimated by the displacements measured by GPS data.

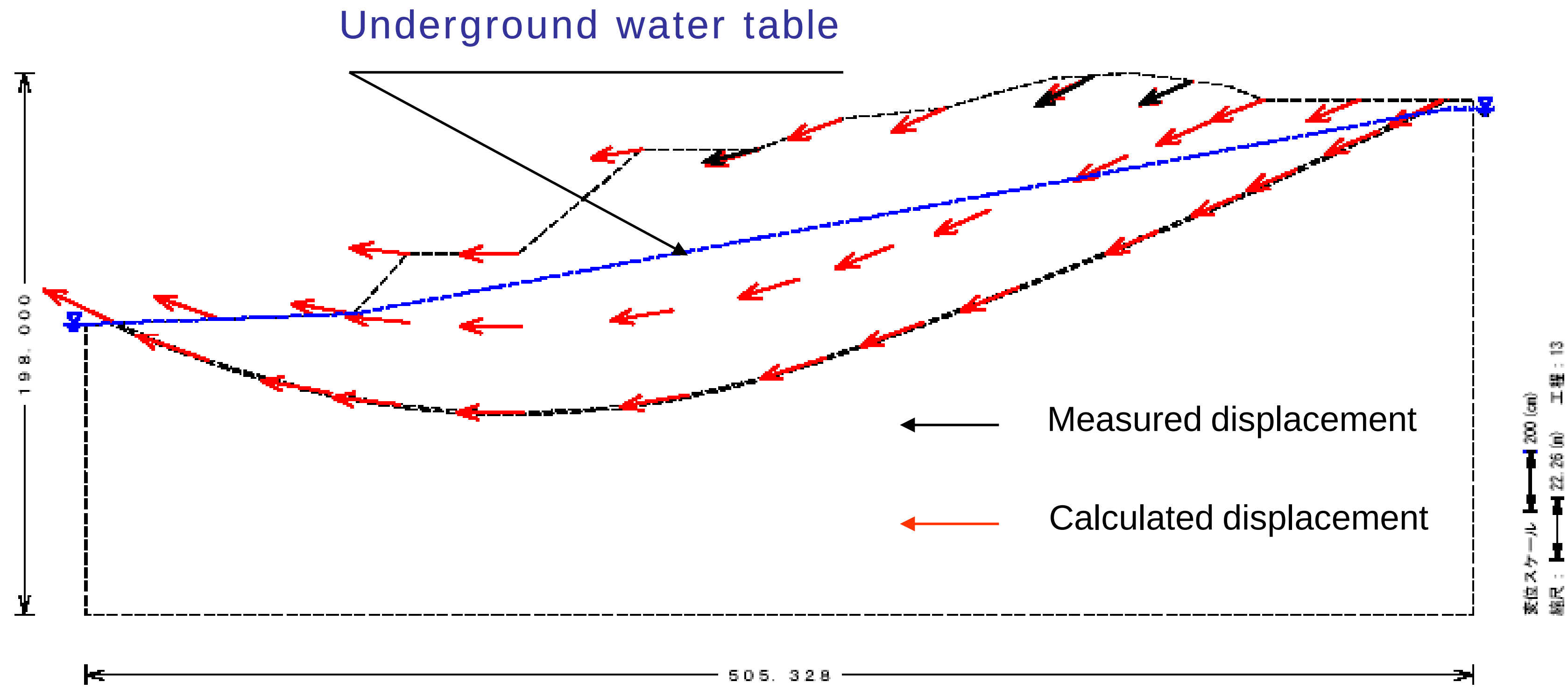
Two potential sliding planes (Case 1 and Case 2) are assumed for back analysis.



Analysis regions of numerical model (Case 1)
Thickness of sliding plane t = 1m



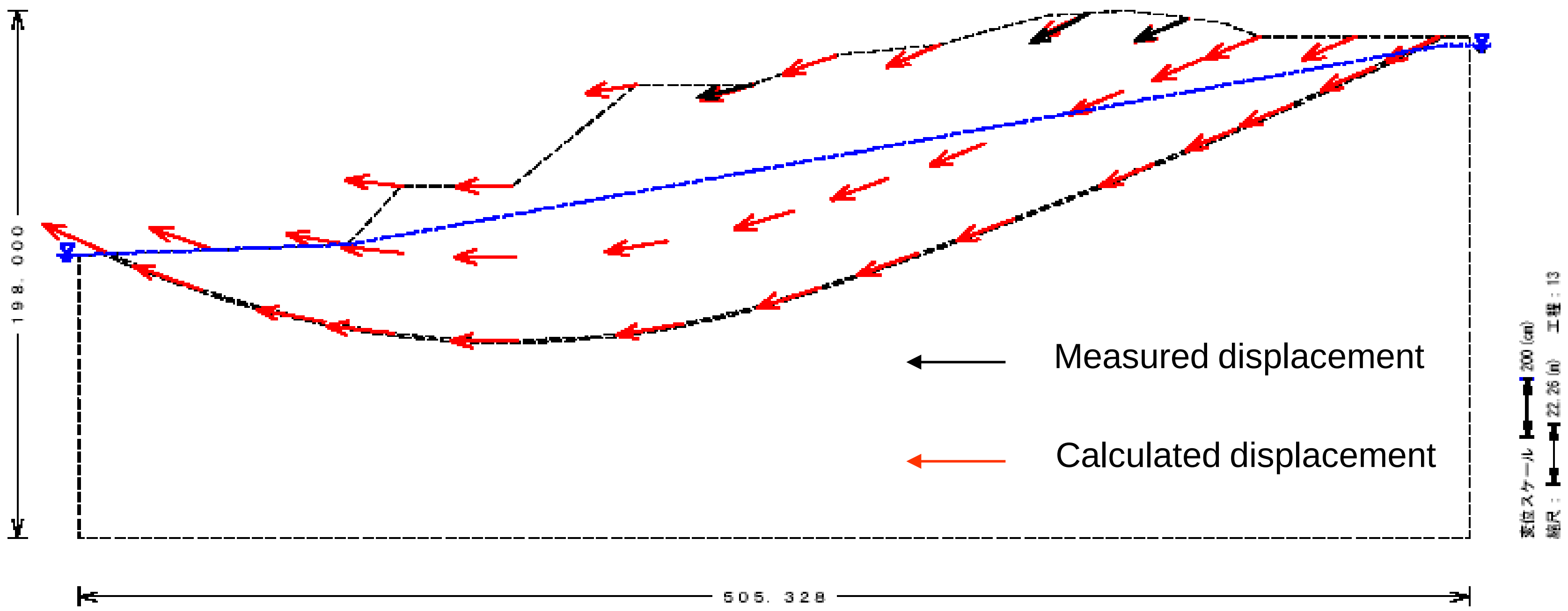
Displacement vectors (Case 1)
Thickness of sliding plane t = 1m,
Back analysis results;
E = 28,000 kN/m², C = 147 kN/m², φ = 30 ~ 15°, m = 0.0118,
Fs = 1.073



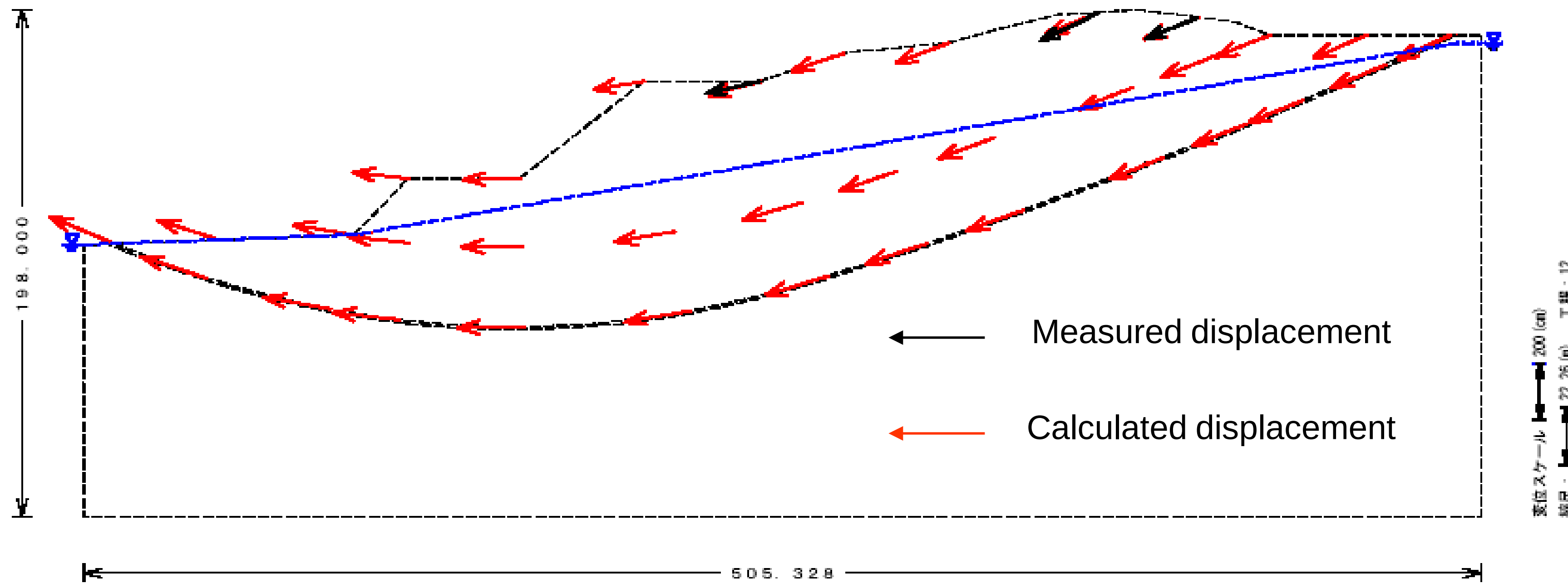
Displacement vectors (Case 1)
Thickness of sliding plane t = 1m,
Back analysis results;

E = 56,000 kN/m², C = 241 kN/m², φ = 30 ~ 15°, m = 0.0060,

Fs = 1.004

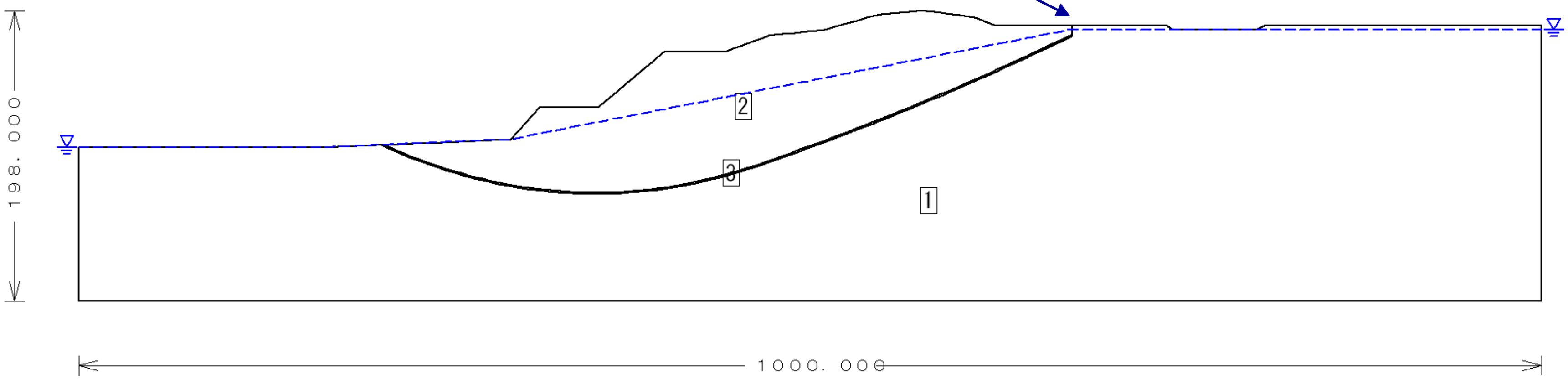


Displacement vectors (Case 1)
Thickness of sliding plane t = 1m,
Back analysis results;
E = 112,000 kN/㎡, C = 395 kN/㎡, φ = 30 ~ 15°, m = 0.0031,
Fs = 0.959

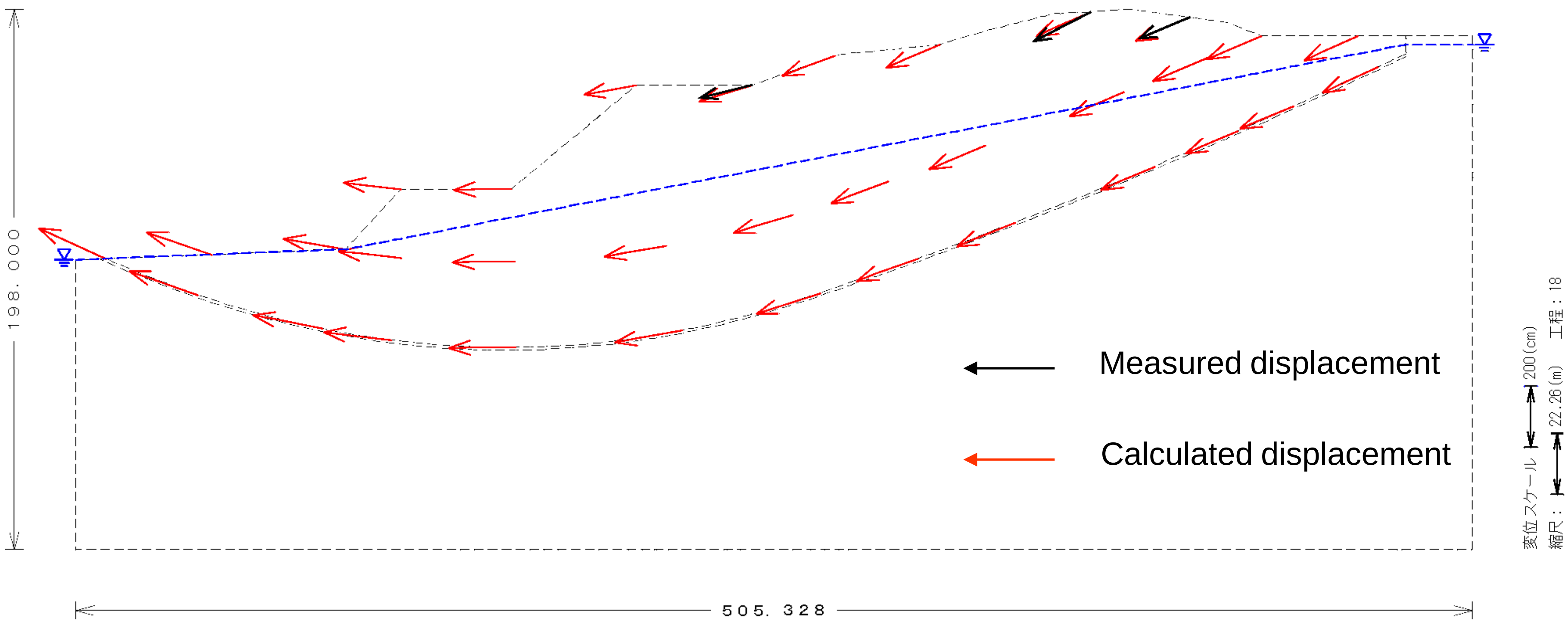


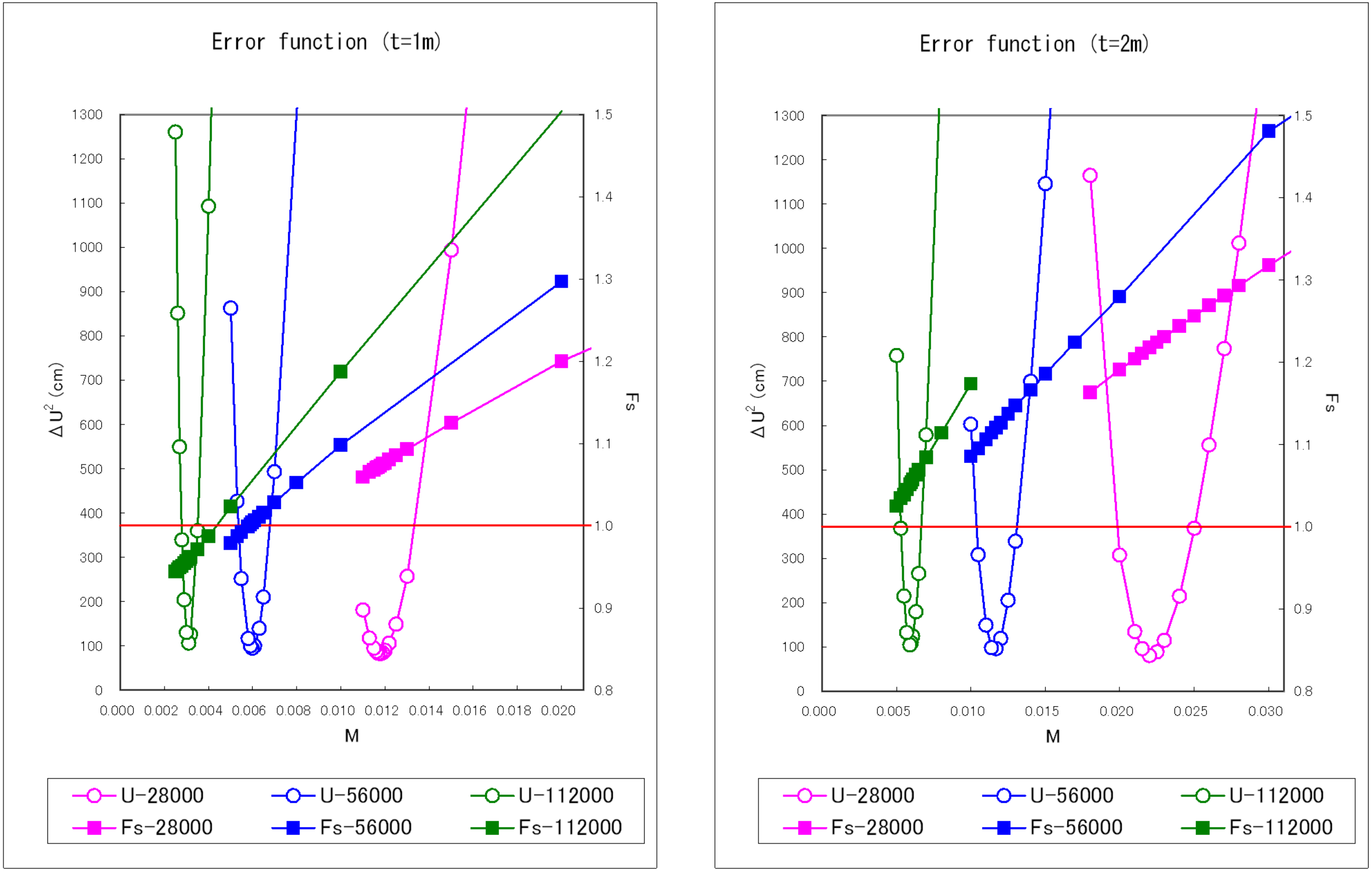
No-tension analysis (Case 1)
Thickness of sliding plane t = 1m

In order to cut off tensile stress occurred here,
8 m deep crack was placed.



No-tension analysis; Displacement vectors (Case 1)
Thickness of sliding plane t = 1m
Back analysis results;
E = 56000 kN/m2, C = 241 kN/m2, φ = 30 ~ 15, m = 0.0060,
Fs = 0.993





Relationship among m-value, error function and safety factor of sliding surface (Case No.1)

Comparison between the measured and calculated displacements (Case 1)
Back-analyzed anisotropic parameter *m*, and factor of safety (Fs)

Thickness of sliding plane t (m)	E (kN/m2)	M	Fs	Δ U2	δ x(6/4) −168.0	δ z(6/4) −46.0	δ x(2/67) −183.3	δ z(2/67) −96.3	δ x(2/68) −164.4	δ z(2/68) −69.8	STEP
1.0	28000	0.0118	1.073	83	170.0	53.0	172.1	82.8	173.2	77.4	13
	56000	0.0060	1.004	95	169.5	52.7	173.3	79.9	174.1	77.4	13
	112000	0.0031	0.959	107	169.2	52.3	174.5	77.8	175.2	77.7	12
2.0	28000	0.0220	1.218	81	170.9	53.5	172.2	85.6	172.8	80.3	15
	56000	0.0117	1.120	95	167.3	52.0	171.4	79.6	172.3	77.1	14
	112000	0.0059	1.051	105	169.5	52.5	174.4	78.5	175.1	78.3	14

$$\Delta \boldsymbol{U}^2 = \sum_{i=1}^{M} (u_i^c - u_i^m)^2 \rightarrow \min.$$

Comparison between the measured and calculated displacements (Case 2)
Back-analyzed anisotropic parameter *m*, and factor of safety (Fs)

Thickness of sliding surface t (m)	E (kN/m2)	M	Fs	ΔU2	δx(6/4) −168.0	δz(6/4) −46.0	δx(2/67) −183.3	δz(2/67) −96.3	δx(2/68) −164.4	δz(2/68) −69.8	STEP
1.0	28000	0.006	1.102	212	180.1	30.2	175.5	83.6	178.6	91.0	16
	56000	0.003	1.048	201	177.2	30.5	171.2	82.7	173.4	91.4	16
	112000	0.0015	1.009	200	176.2	31.1	169.3	82.2	171.0	91.6	17

Non-tension analysis (case 1)
Comparison between the measured and calculated displacements
Back-analyzed anisotropic parameter *m*, and factor of safety (Fs)

Thickness of Sliding plane (m)	E (kN/m2)	M	Fs	ΔU2	δx(6/4) -168.0	δz(6/4) -46.0	δx(2/67) -183.3	δz(2/67) -96.3	δx(2/68) -164.4	δz(2/68) -69.8	STEP
1.0	56000	0.0060	0.993	99	171.7	53.3	176.2	80.3	176.9	77.9	18

Conclusions for Case Study

- The strength parameters such as cohesion and internal friction angle of the materials were determined from the measured displacements, and the factor of safety was calculated by using them.

- The factor of safety becomes nearly 1.0 just before failure occurred. This means that the strength parameters determined by the back analysis are reasonable.

- In conclusion, the back analysis method can predict the time when the slope will fail only from the surface displacements measured by GPS.

Conclusions

1. Displacement measurements are easy, reliable and economical in both tunnel and slope engineering practice.
2. Back analysis is a powerful tool for assessing the measured displacements.
3. Anisotropic parameter *m* is well applicable in back analysis in observational methods for both tunnels and slopes.
4. Both critical strain and critical shear strain are a very useful in assessing the stability of tunnels and slopes.

Thank you for your attention.