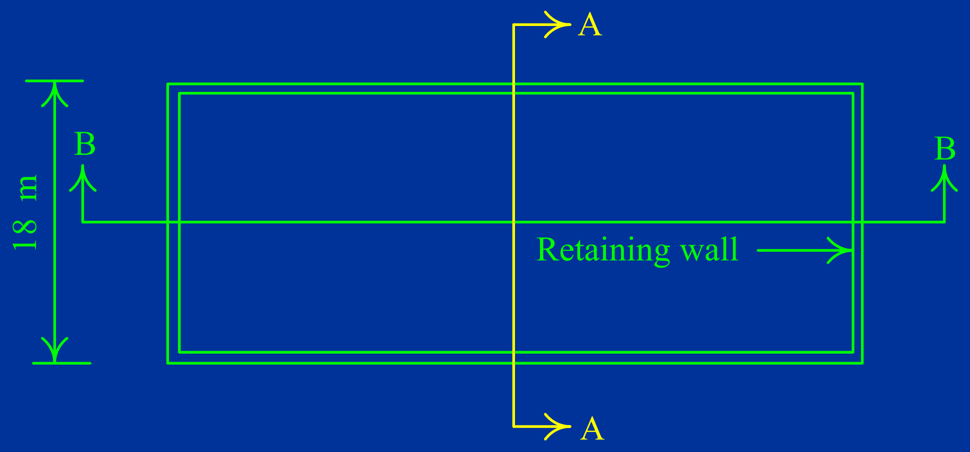
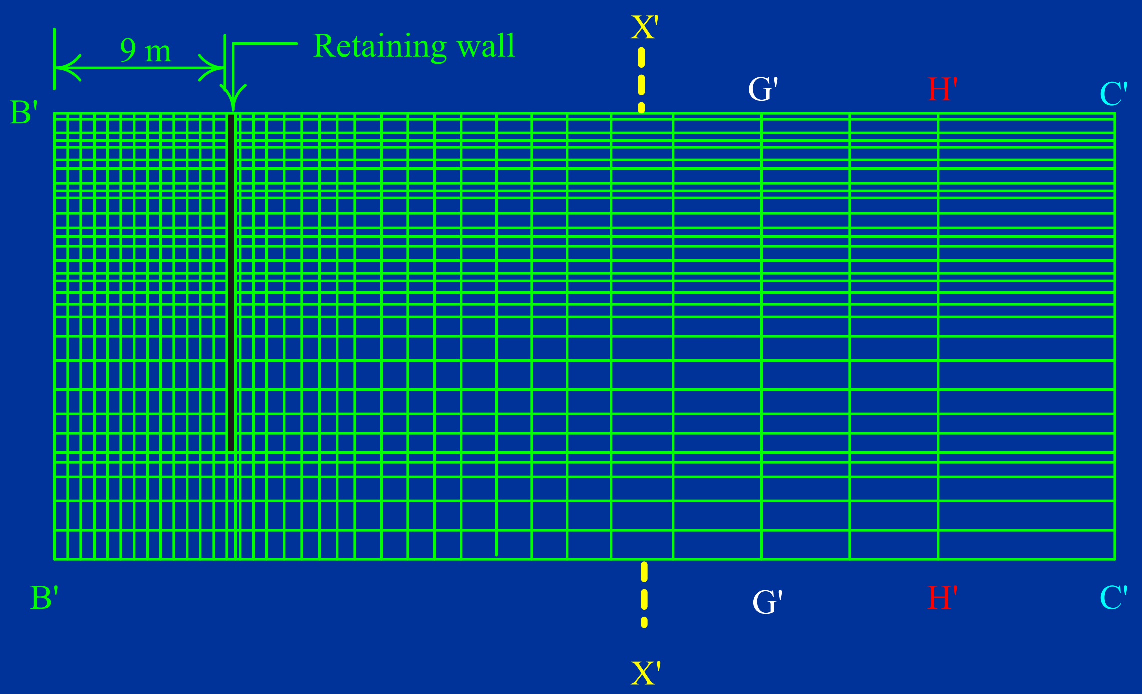
Part II: Finite Element Method

1. Basic principles





Commercial Software:

CRISP

PLAXIS

FLAC

ABAQUS

2. Types of analysis

Effective stress analysis:

All of the calculation in the computer program are based on the effective stress. Soil and water are treated as different material.

Input parameters: c', ϕ', E', ν'

Total stress analysis:

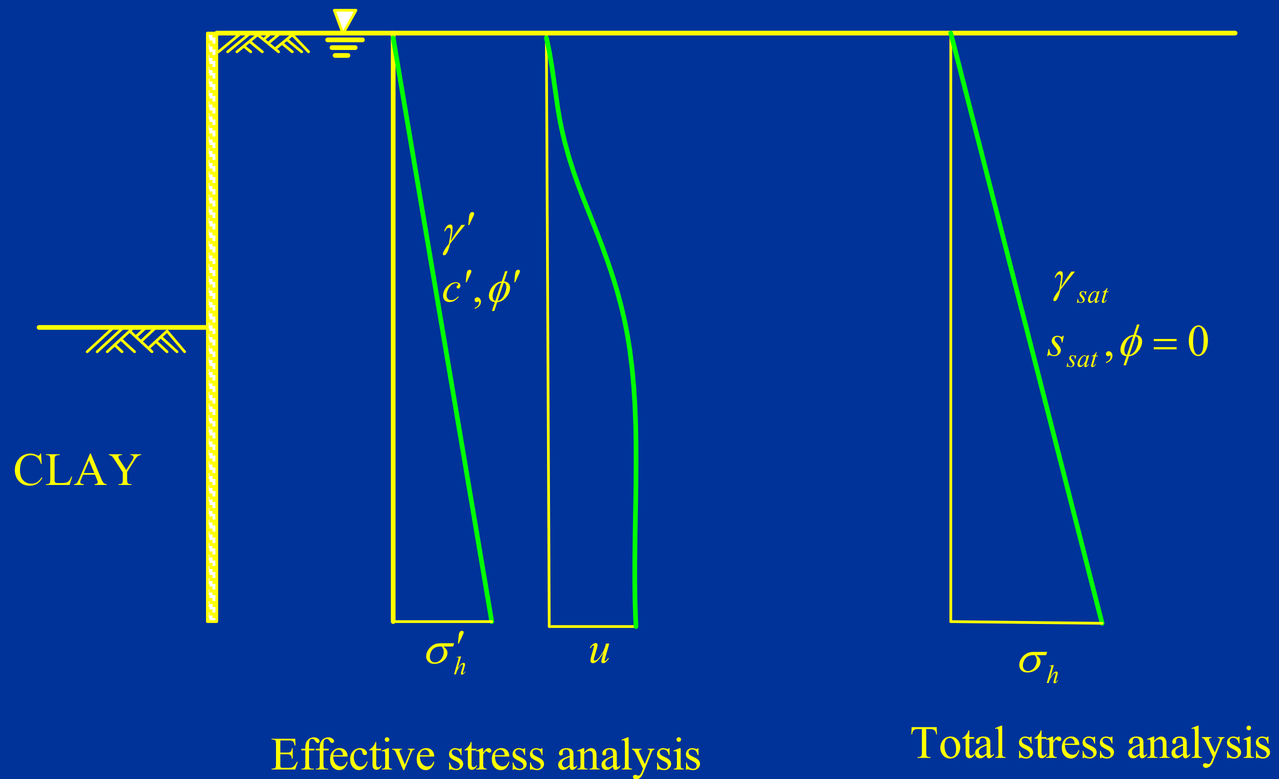
All of the calculation in the computer program are based on the total stress. Only one material, soil-water mixture, exists.
No excess porewater is generated in the program.

Input parameters: $s_u, \phi = 0, E_u, \nu_u$

Sand (drained behavior)--------Effective stress analysis

Clay (undrained behavior)-----Effective stress analysis
-----Total stress analysis

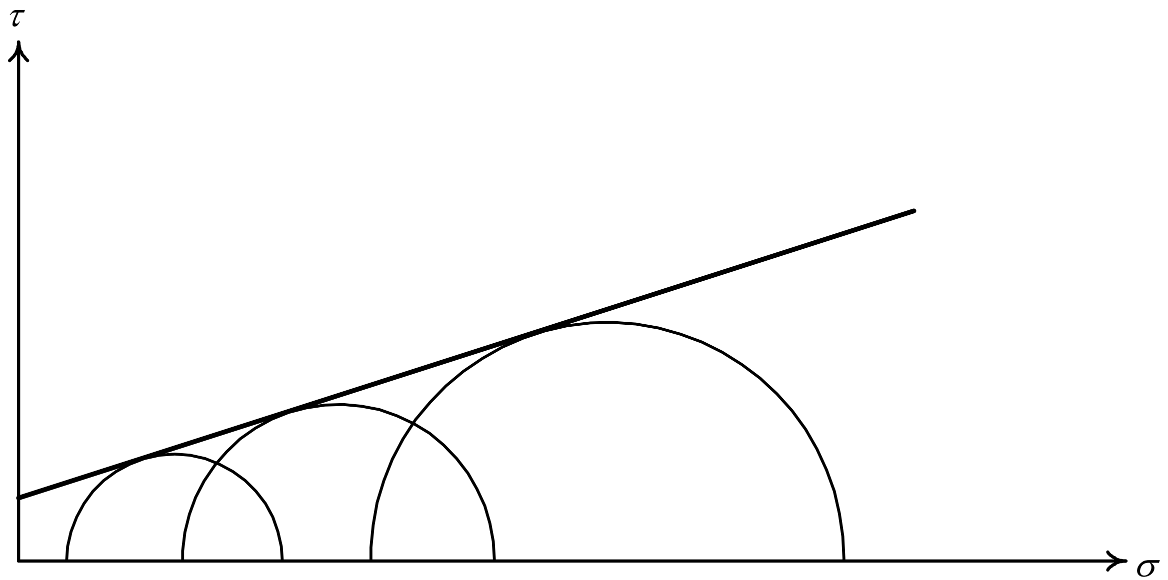
Effective stress undrained analysis:
Total stress undrained analysis:



Effective stress analysis and total stress analysis of clay

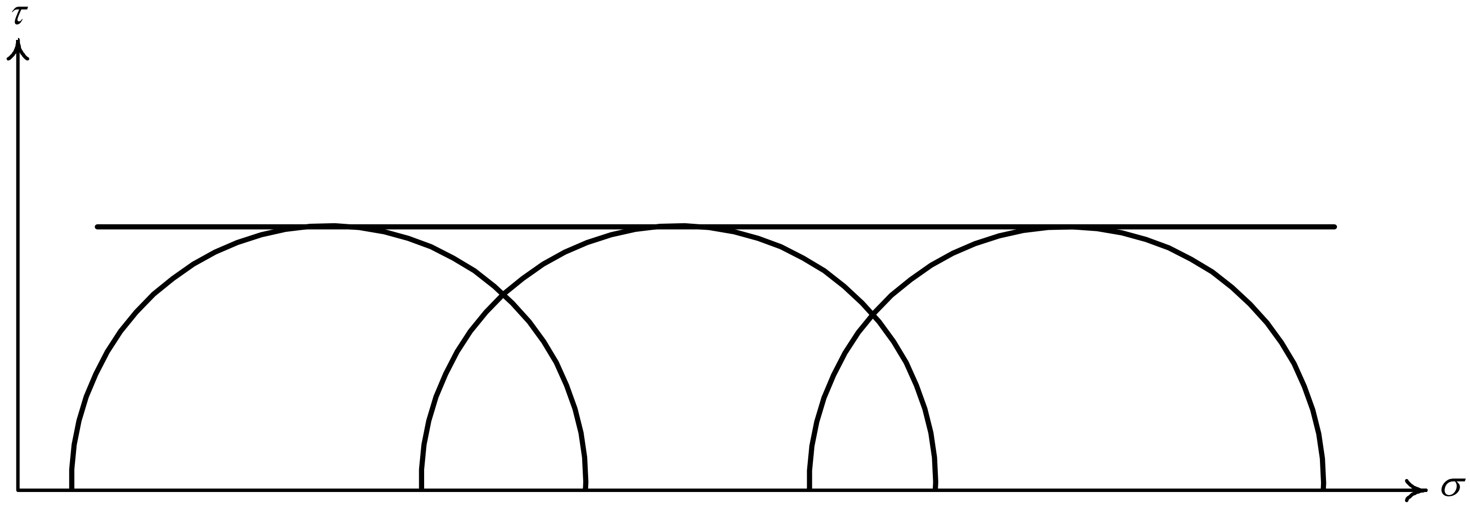
Effective stress analysis:

Effective stress soil model: MC model ($\phi \neq 0$), Cam-clay model



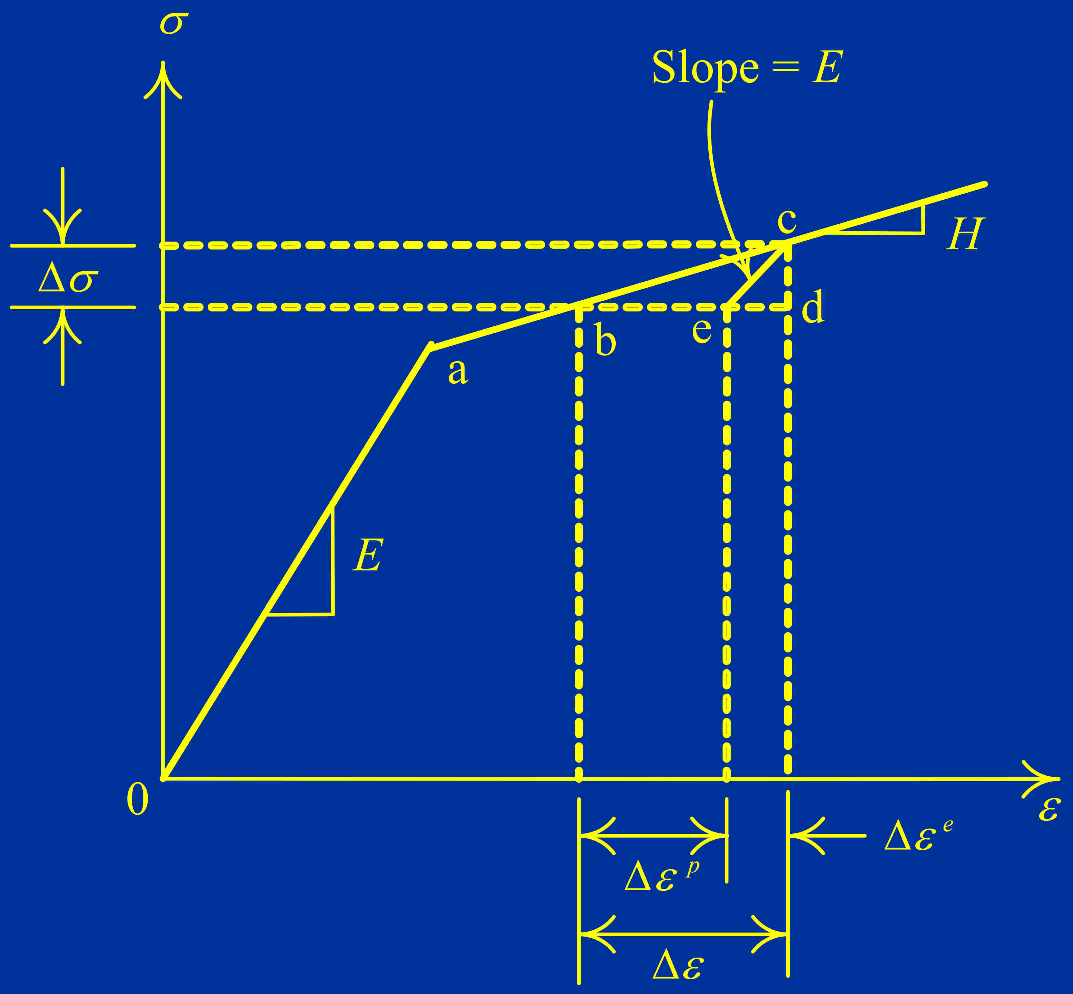
Total stress analysis:

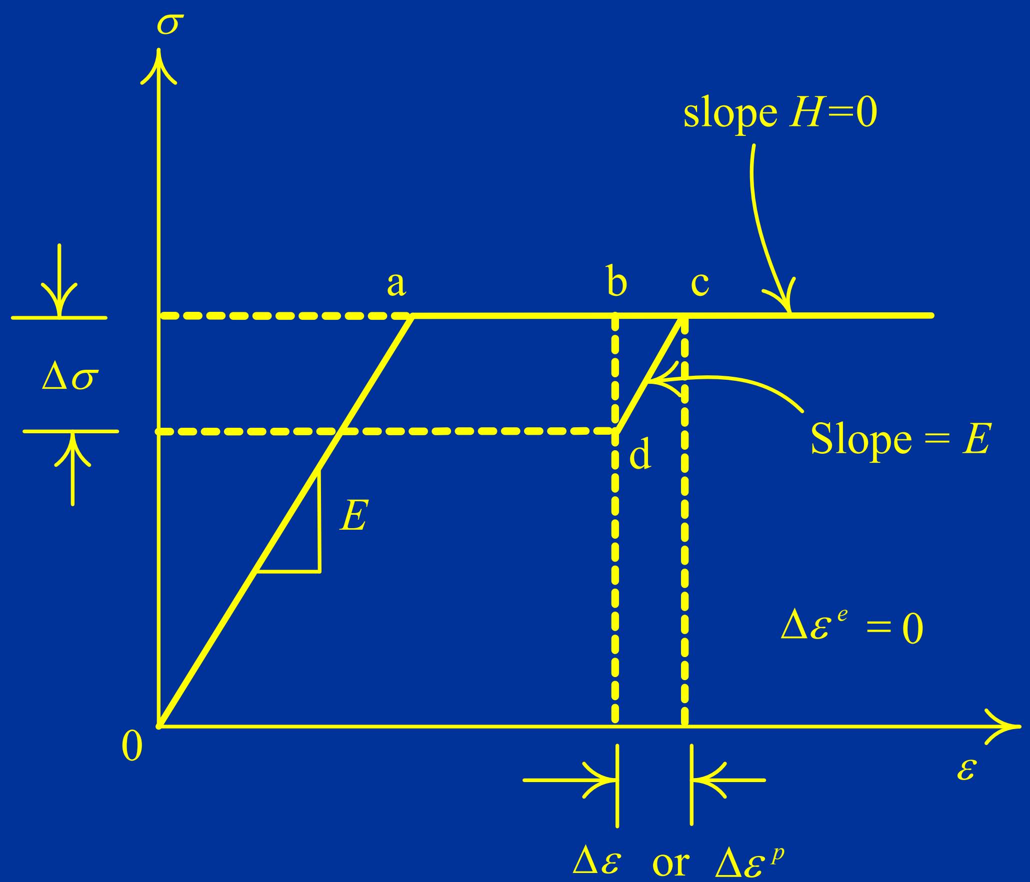
Total stress soil model: MC model (ϕ=0)



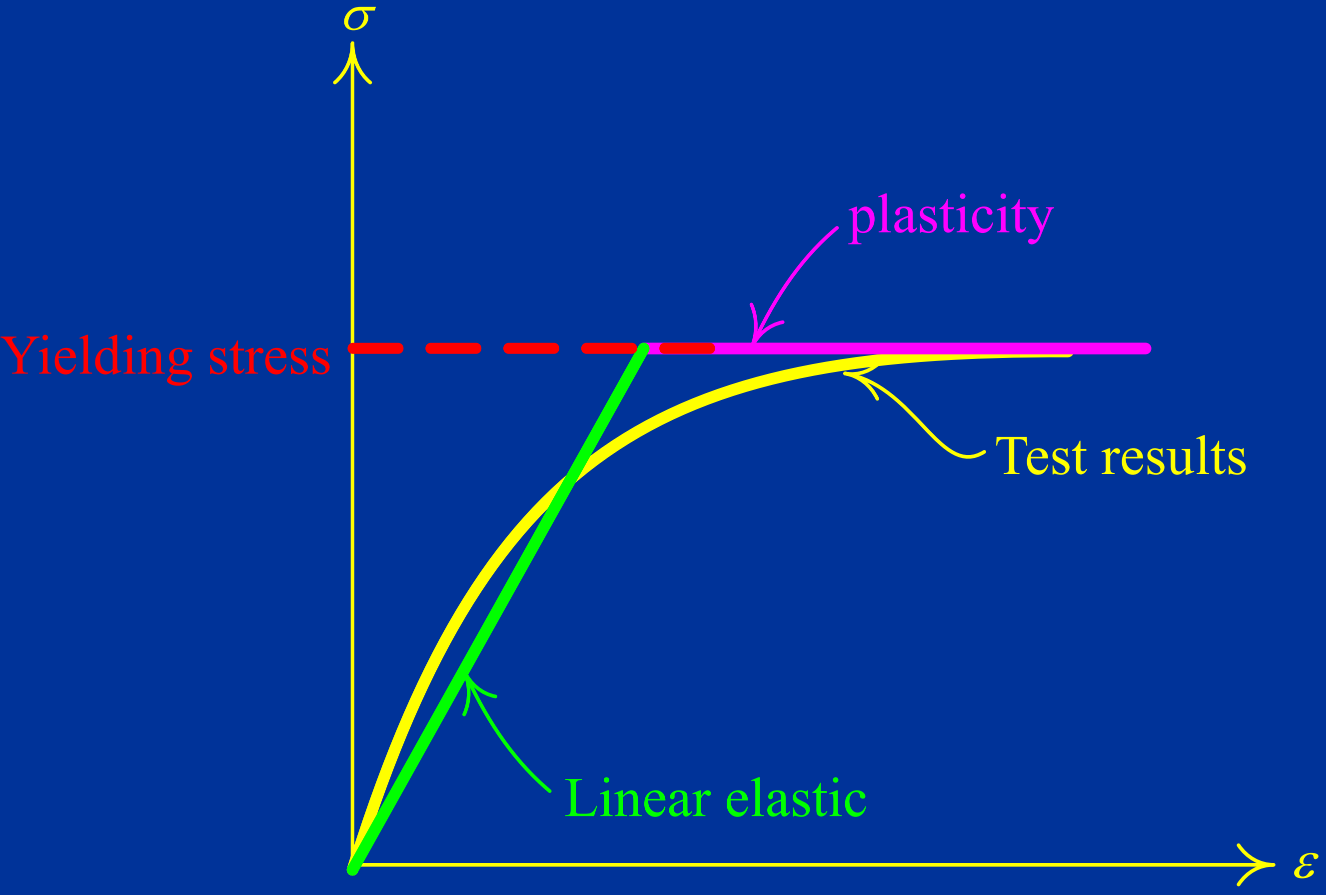
3. Constitutive soil models

Linear elastic elastoplastic model





linear elastic perfectly plastic model



σ
plasticity
Yielding stress
Test results
Linear elastic
ε

8.9.2 Parameters for the hyperbolic Model

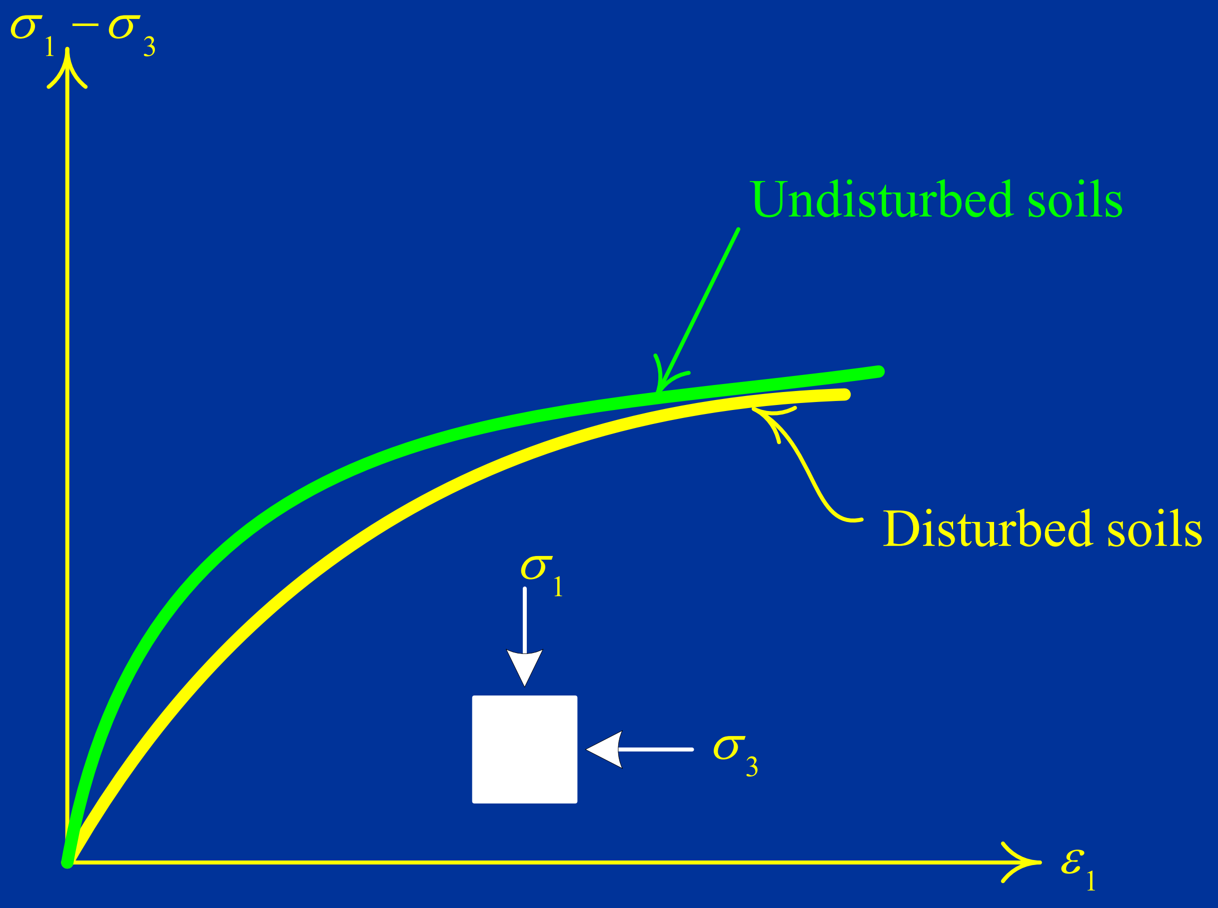


FIGURE 8.32 Stress-strain curves for undisturbed and disturbed soils

TABLE 8.3 Empirical equations for E_s (Bowles, 1988)

Soil type	SPT-N (kPa)	CPT (same unit as q_c)
Sand (normally consolidated)	$E_s = 500(N+15)$ $E_s = (15{,}000 \sim 22{,}000)\ln N$ $E_s = (35{,}000 \sim 50{,}000)\log N$	$E_s = (2 \sim 4)q_c$ $E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N+15)$	
Sand (overconsolidated)	$E_s = 18{,}000 + 750N$	$E_s = (6 \sim 30)q_c$
Gravelly sand and gravel	$E_s = 1{,}200(N+6)$ $E_s = 600(N+15) \quad N \le 15$ $E_s = 600(N+15) + 2{,}000 \quad N > 15$	
Clayey sand	$E_s = 320(N+15)$	$E_s = (3 \sim 6)q_c$
Silty sand	$E_s = 300(N+6)$	$E_s = (1 \sim 2)q_c$
Soft clay		$E_s = (3 \sim 8)q_c$

TABLE 8.2 Range of the Poisson's ratio

Soil type	ν_s
Saturated clay (undrained)	0.5
Unsaturated clay (undrained)	0.35 ~ 0.4
Silty sand	0.3 ~ 0.4
Sand, gravel	0.15 ~ 0.35
Silt	0.3 ~ 0.35
Rock	0.1 ~ 0.4 (depending on type of rock)
Ice	0.36
Concrete	0.15

TABLE 8.4 Ranges of E_s for various soils (Bowles, 1988)

Soil type	E_s (MPa)
Very soft clay	2 ~ 15
Soft clay	5 ~ 25
Medium stiff clay	15 ~ 50
Stiff clay	50 ~ 100
Sandy clay	25 ~ 250
Silty sand	5 ~ 20
Loose sand	10 ~ 25
Dense sand	50 ~ 81
Loose gravel	50 ~ 150
Dense gravel	100 ~ 200
Shale	150 ~ 5000
Silt	2 ~ 20

Elastic incremental model---the hyperbolic model

Duncan and Chang's model

References:

Duncan, J.M. and Chang, Y.Y. (1970), Nonlinear analysis of stress and strain in soils, Journal of the Soil Mechanics and Foundation Division, ASCE, Vo. 96, No. 5.

Wong, K.S and Duncan, JM (1974), Hyperbolic Stress-Strain Parameters for Nonlinear Finite Element Analysis of Stresses and Movements in Soil, Department of Civil Engineering, University of California, Berkeley.

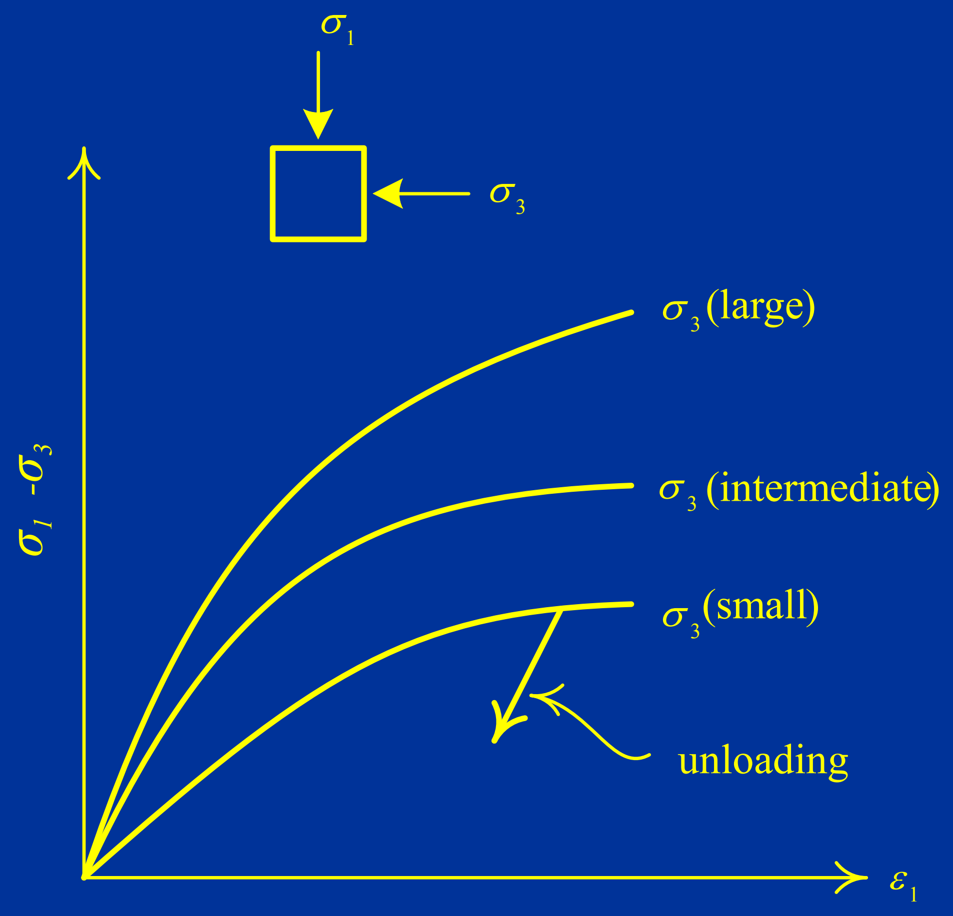
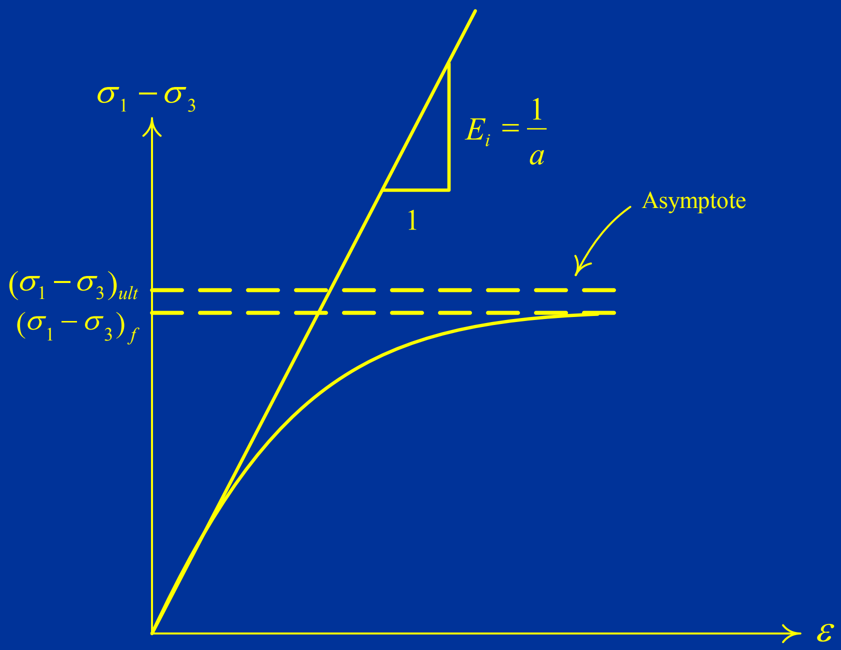


FIGURE 8.10 Typical stress-strain relations of soils



Konder (1963) proposed:

$$\sigma_1 - \sigma_3 = \frac{\varepsilon}{a + b\varepsilon}$$

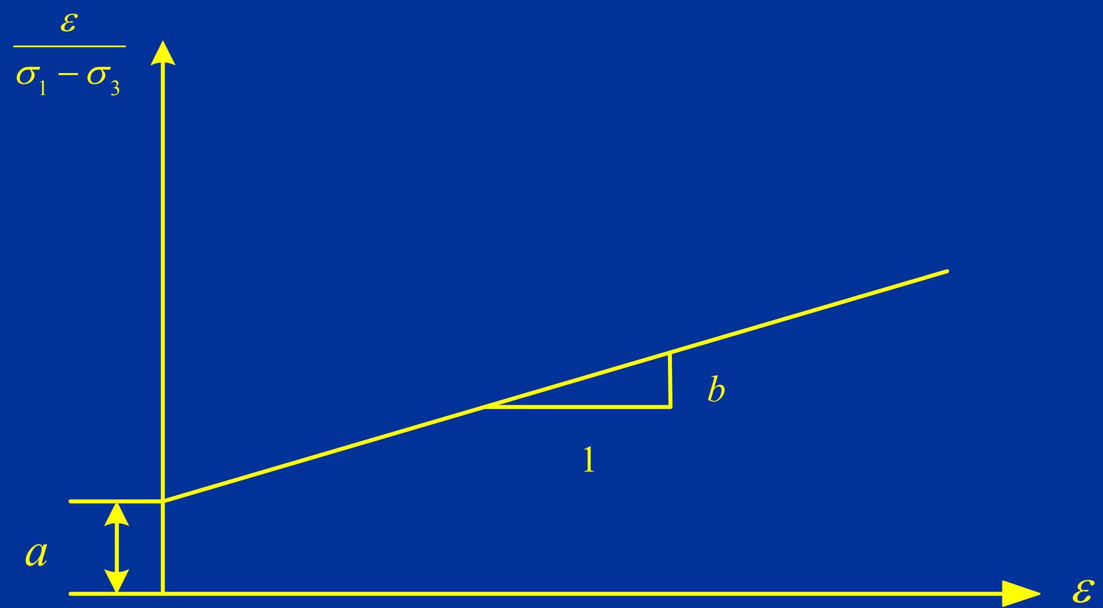
$$(\sigma_1 - \sigma_3)_{ult} = \frac{1}{b} \qquad E_i = \frac{1}{a}$$

$$\sigma_1 - \sigma_3 = \frac{\varepsilon}{\frac{1}{E_i} + \frac{\varepsilon R_f}{(\sigma_1 - \sigma_3)_f}}$$

$$(\sigma_1 - \sigma_3)_f = R_f (\sigma_1 - \sigma_3)_{ult}$$

R_f : failure ratio, 0.6~1.0

Transformation $\sigma \sim \varepsilon$ curve:



$$(\sigma_1 - \sigma_3)_{ult} = \frac{1}{b}$$

$$(\sigma_1 - \sigma_3)_f = R_f(\sigma_1 - \sigma_3)_{ult}$$

R_f : failure ratio, 0.6~1.0

$$E_i = \frac{1}{a}$$

$$\sigma_1 - \sigma_3 = \frac{\varepsilon}{\dfrac{1}{E_i} + \dfrac{\varepsilon R_f}{(\sigma_1 - \sigma_3)_f}}$$

$$E_i = f(\sigma_3)$$

Janbu(1963):

$$E_i = KP_a\left(\frac{\sigma_3}{P_a}\right)^n$$

where P_a =atmospheric pressure = 1.033 kg/cm^2 =101.3 kN/ m^2 =116.2 lb/ ft^2

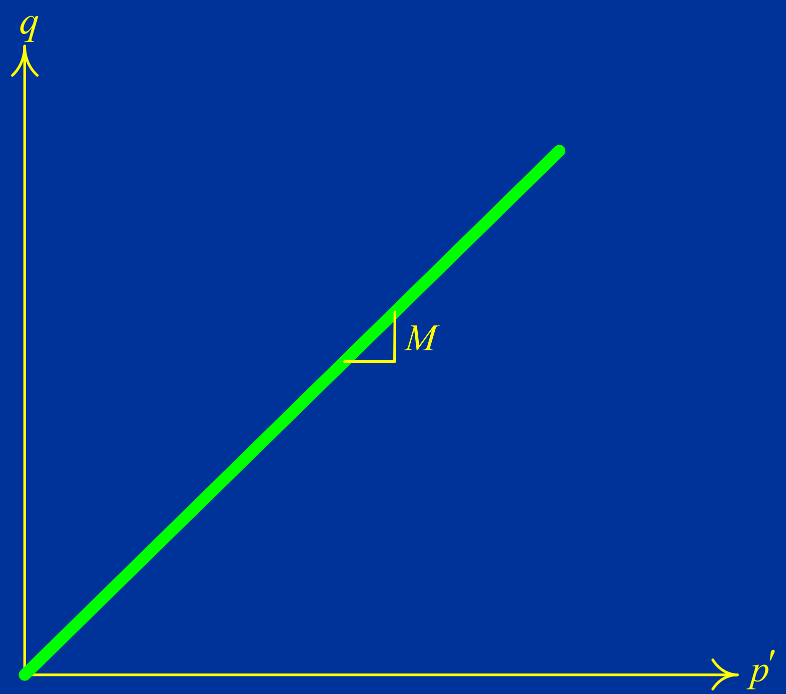
K = dimensionless stiffness modulus number

n = dimensionless stiffness modulus exponent

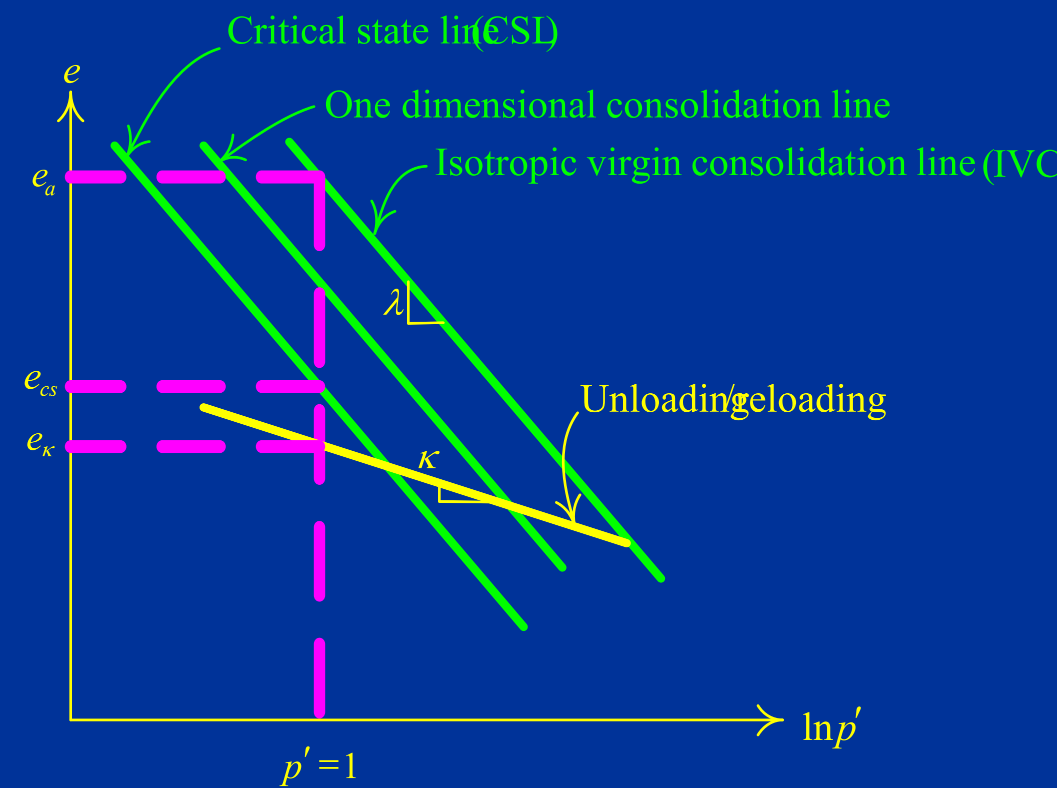
Saturated clay (undrained) $\nu = 0.5$

Parameter required: c , ϕ , R_f , K, n , K_{ur}, ν (1970 version)

Cam-clay and other high order models (MITE3, MITS1)



(a)



(b)

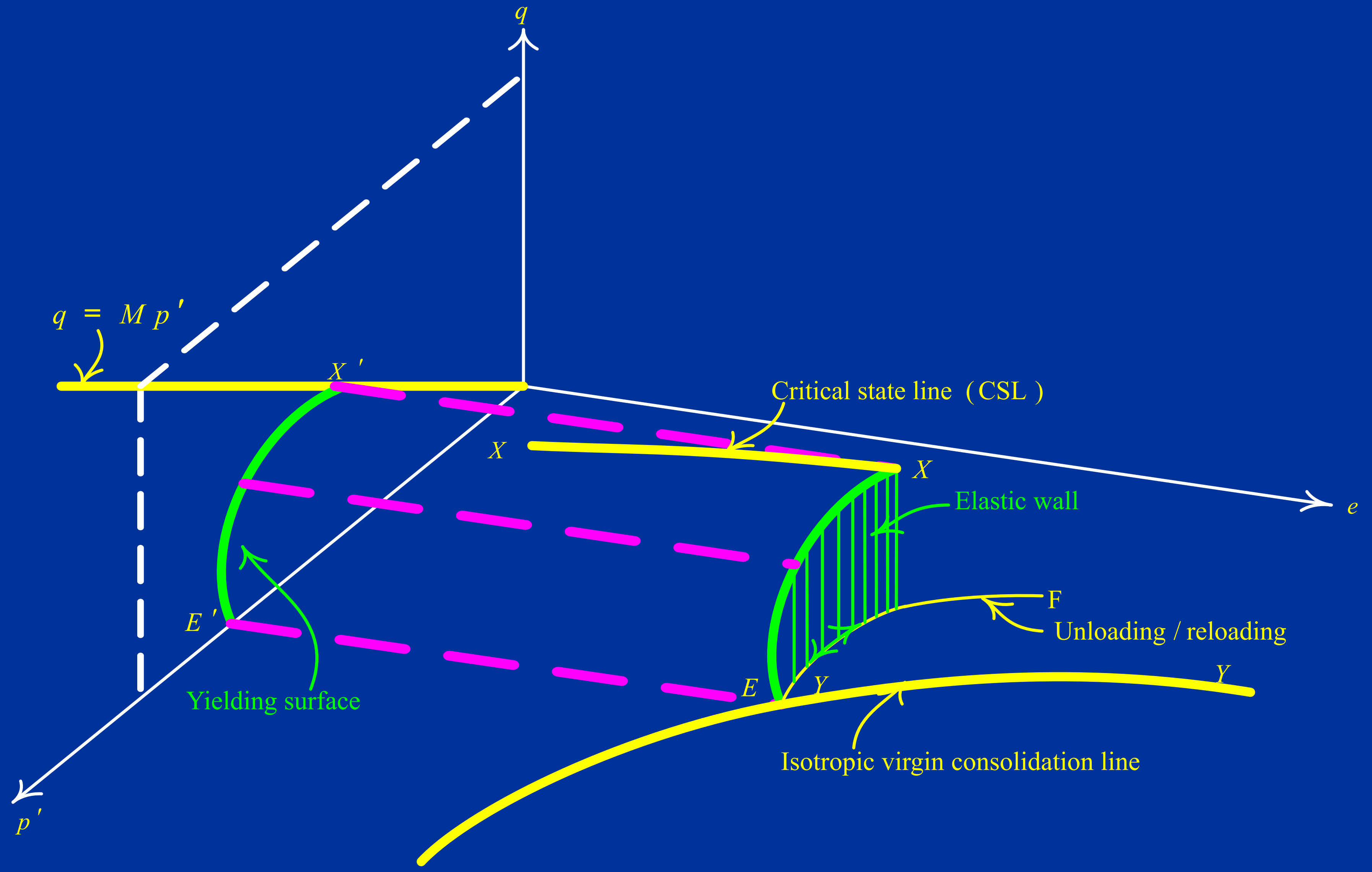


FIGURE 8.18 State boundary surface

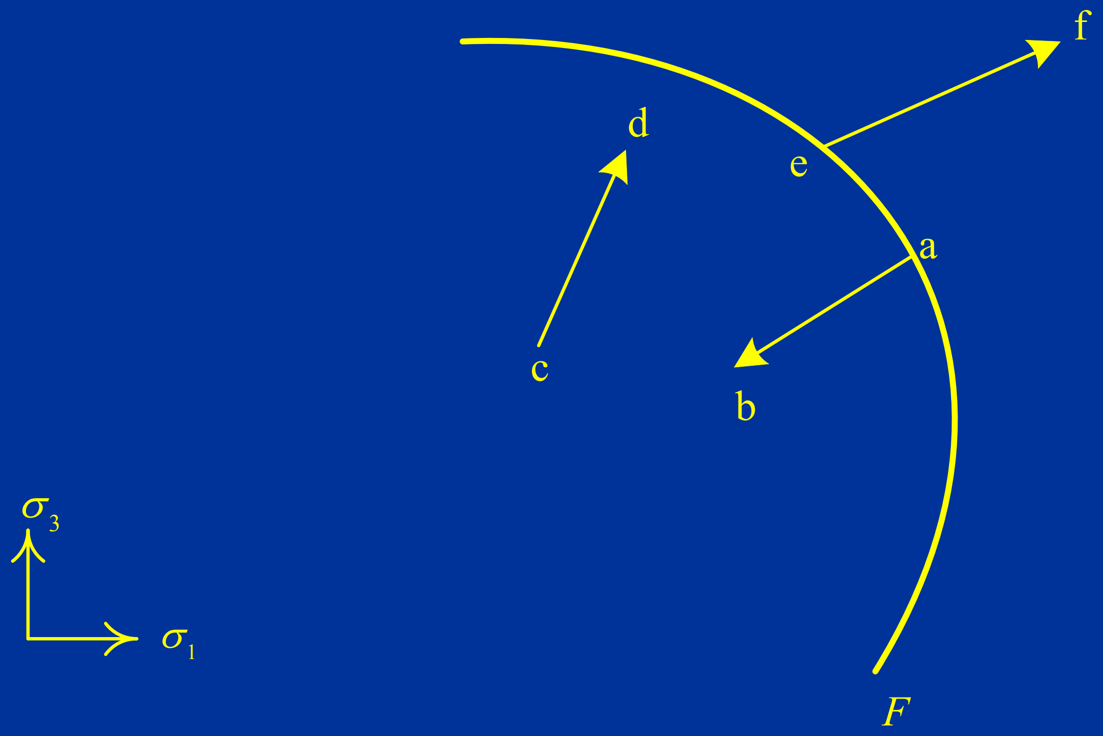


FIGURE 8.16 Yielding surface and stress paths

The equations for the state boundary:

$$\frac{p}{p_e} = \left(\frac{M^2}{M^2 + q^2 / p^2} \right)^{(1-\kappa/\lambda)}$$

$$p_e = \exp\left(\frac{e_a - e}{\lambda} \right)$$

The yielding equation $$p = p_0 \left(\frac{M^2}{M^2 + q^2 / p^2} \right)$$

The Cam-clay model requires the following parameters: ν , E , M , λ and κ.

$$M = \frac{6 - \sin \phi'}{3 - \sin \phi'}$$

$$\lambda = \frac{C_c}{2.303}$$

$$\kappa = \frac{C_s}{2.303}$$

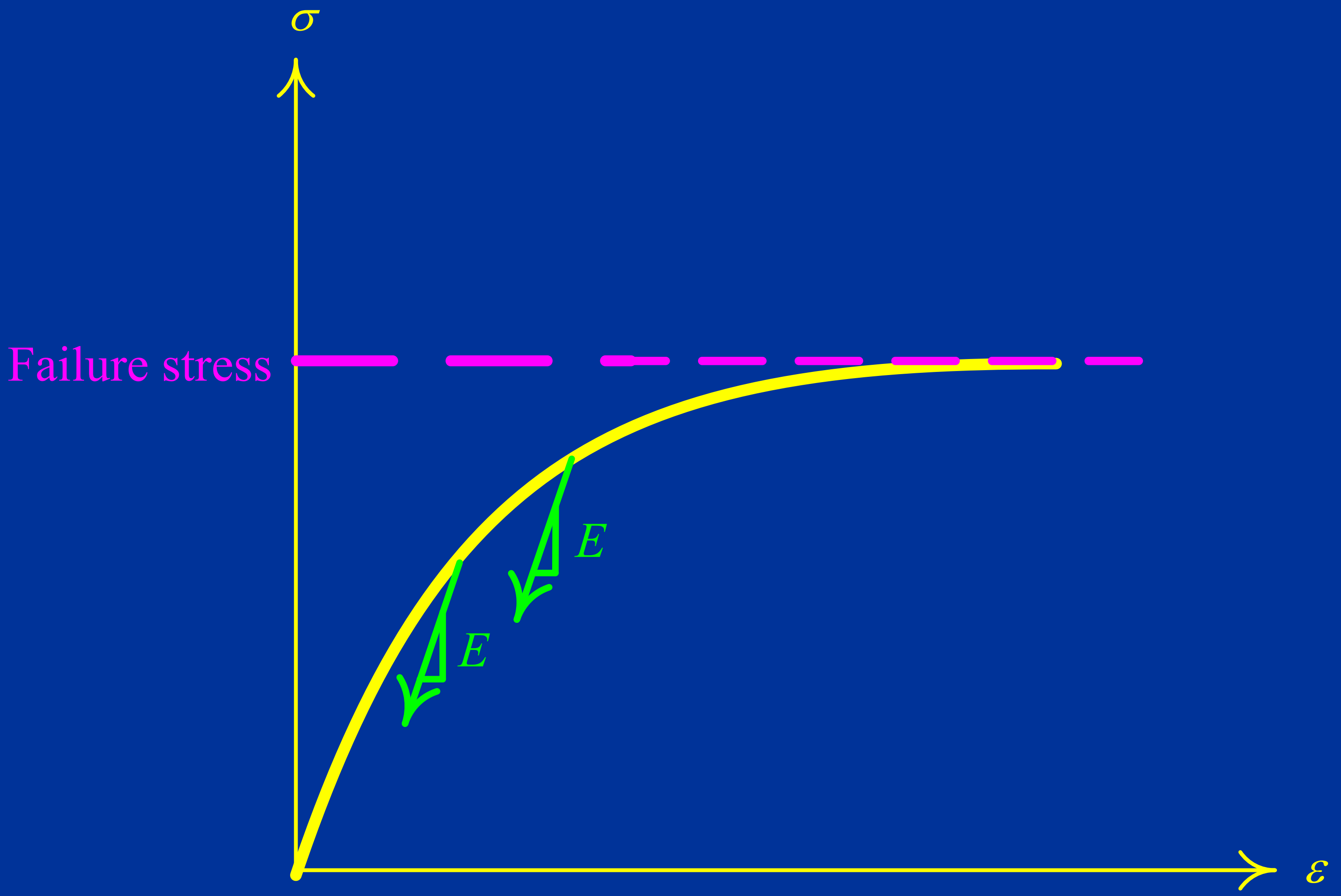


FIGURE 8.34 Stress-strain behavior of soil obtained from the simulation using Cam-Clay elastoplastic model

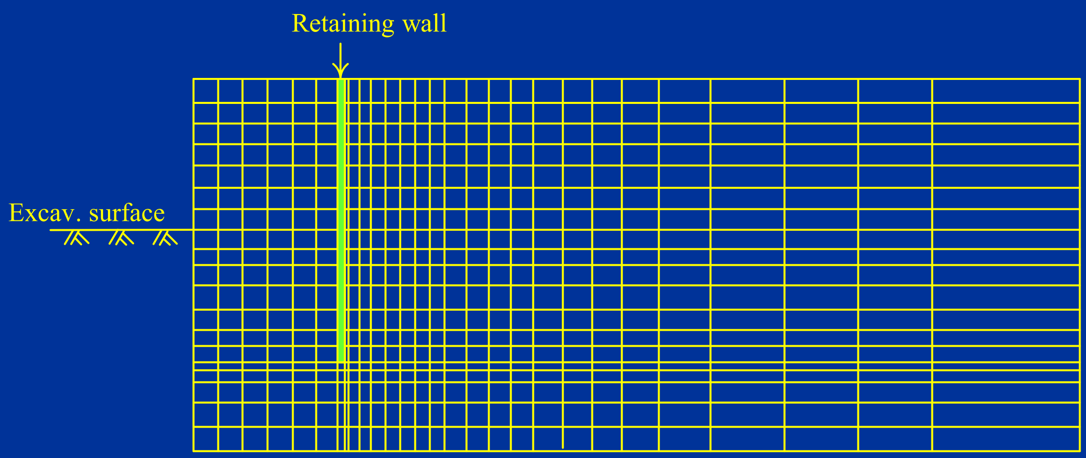
$$e = e_{cs} - \lambda \ln p'$$

$$de = -\kappa \frac{dp'}{p'}$$

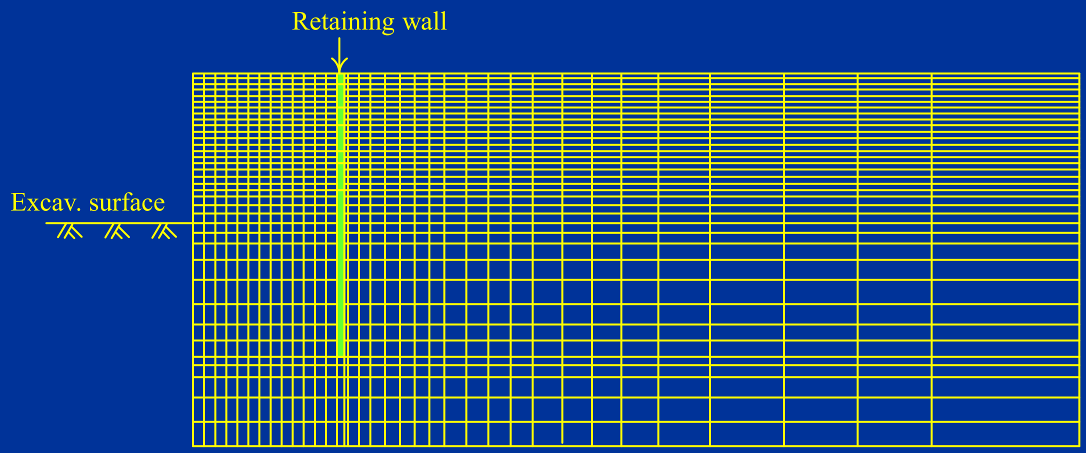
According to the definition of bulk modulus, we can derive the effective bulk modulus (under the drained condition) as:

$$B' = -\frac{dp'}{d\varepsilon_v} = -\frac{dp'}{de/(1+e)} = \frac{(1+e)p'}{\kappa}$$

4. Mesh generation for excavation



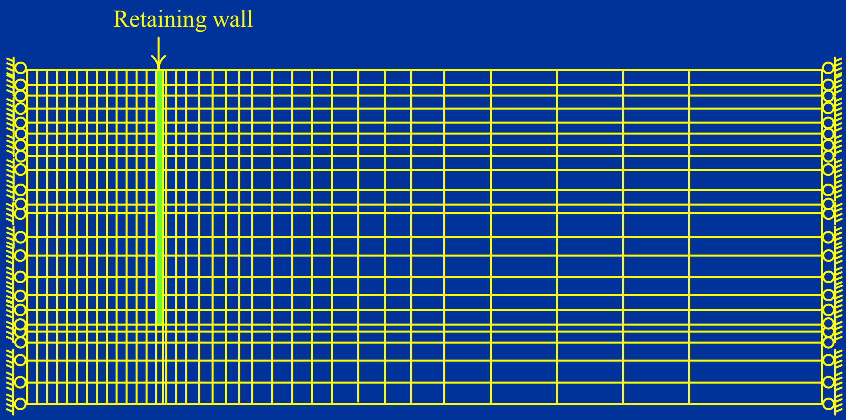
(a)



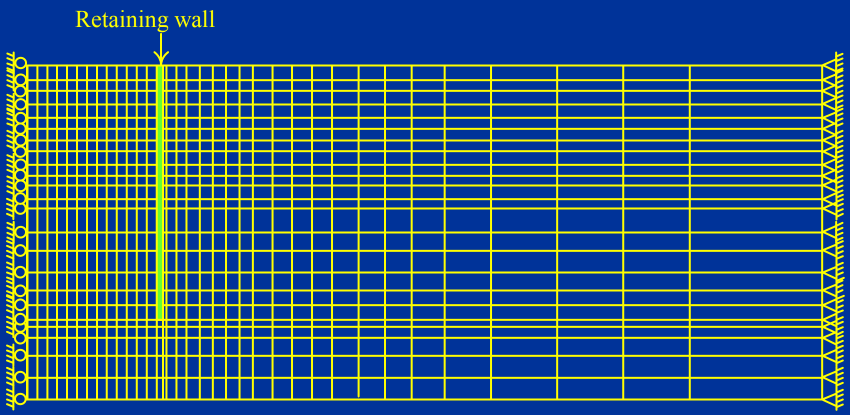
(b)

FIGURE 8.25 The finite element meshes used in the analysis of excavation
(a) bad mesh (b) good mesh

Boundary conditions

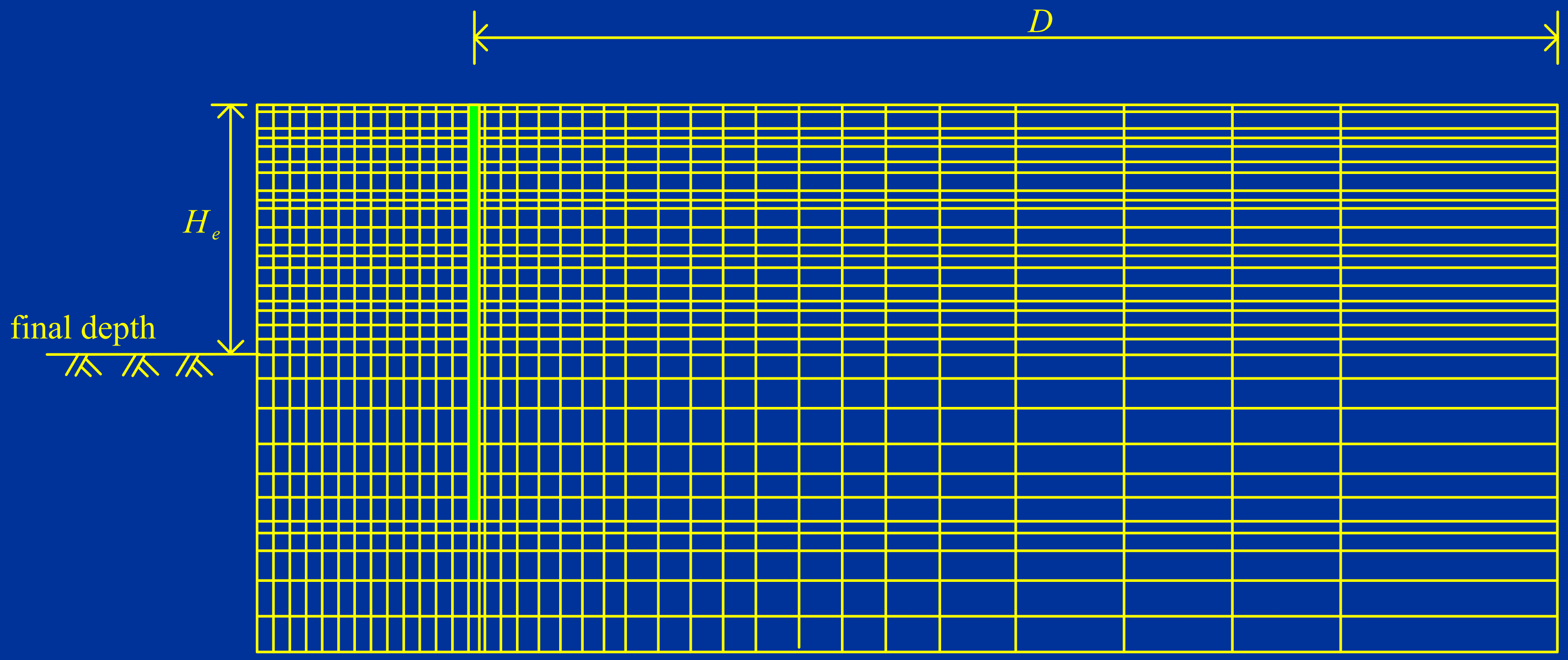


(a)



(b)

FIGURE 8.26 Boundary conditions of the finite element mesh
(a) boundary outside the excavation zone is allocated with rollers
(b) boundary outside excavation zone is allocated with hinges



ground settlement : $D \geq 4H_e$

wall deflection : $D \geq 3H_e$

FIGURE 8.27 Distance of the boundary required for the analysis of wall deflection or ground settlement

5. Corner effect on deformation behavior

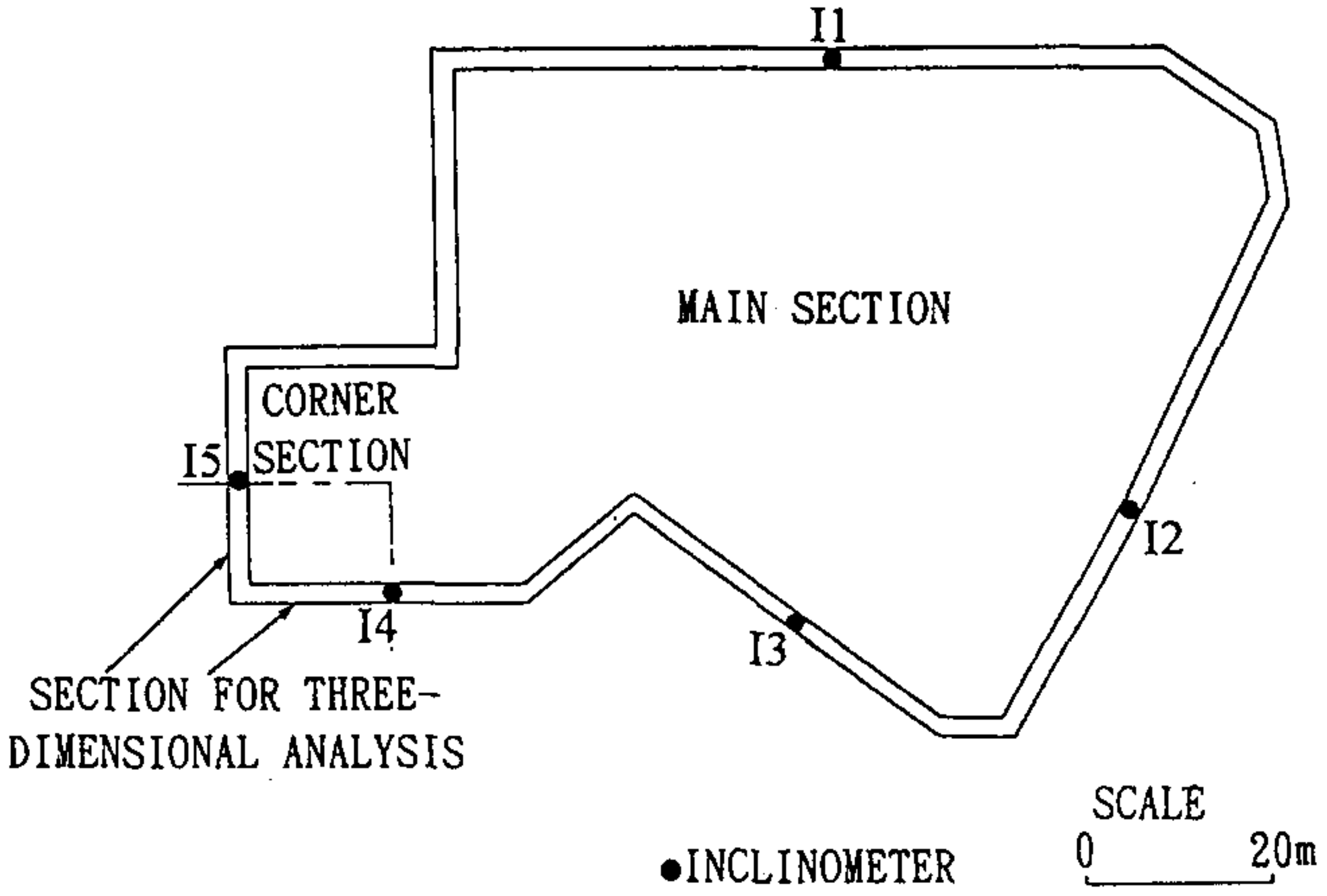


FIG. 14. The Hai-Hua Building Site

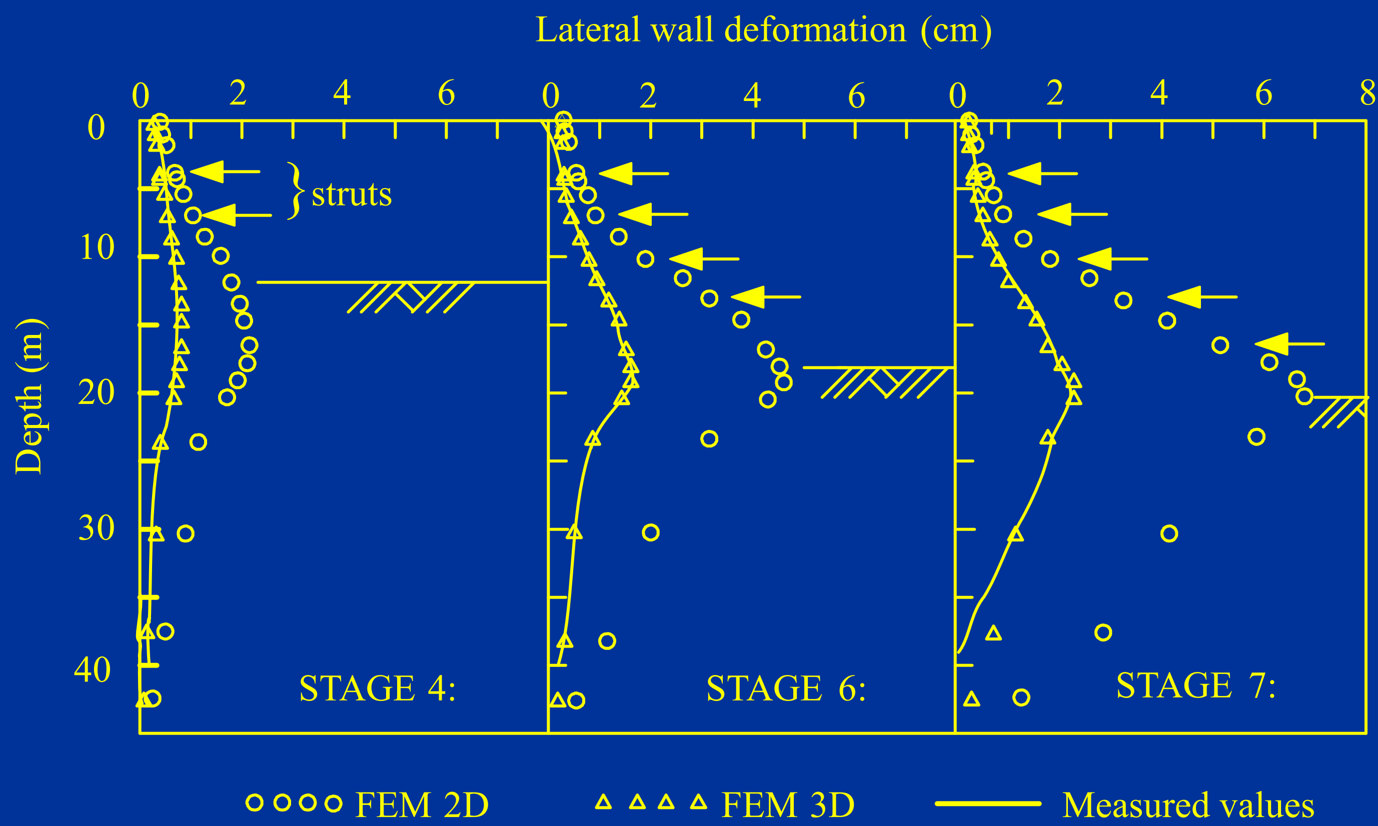
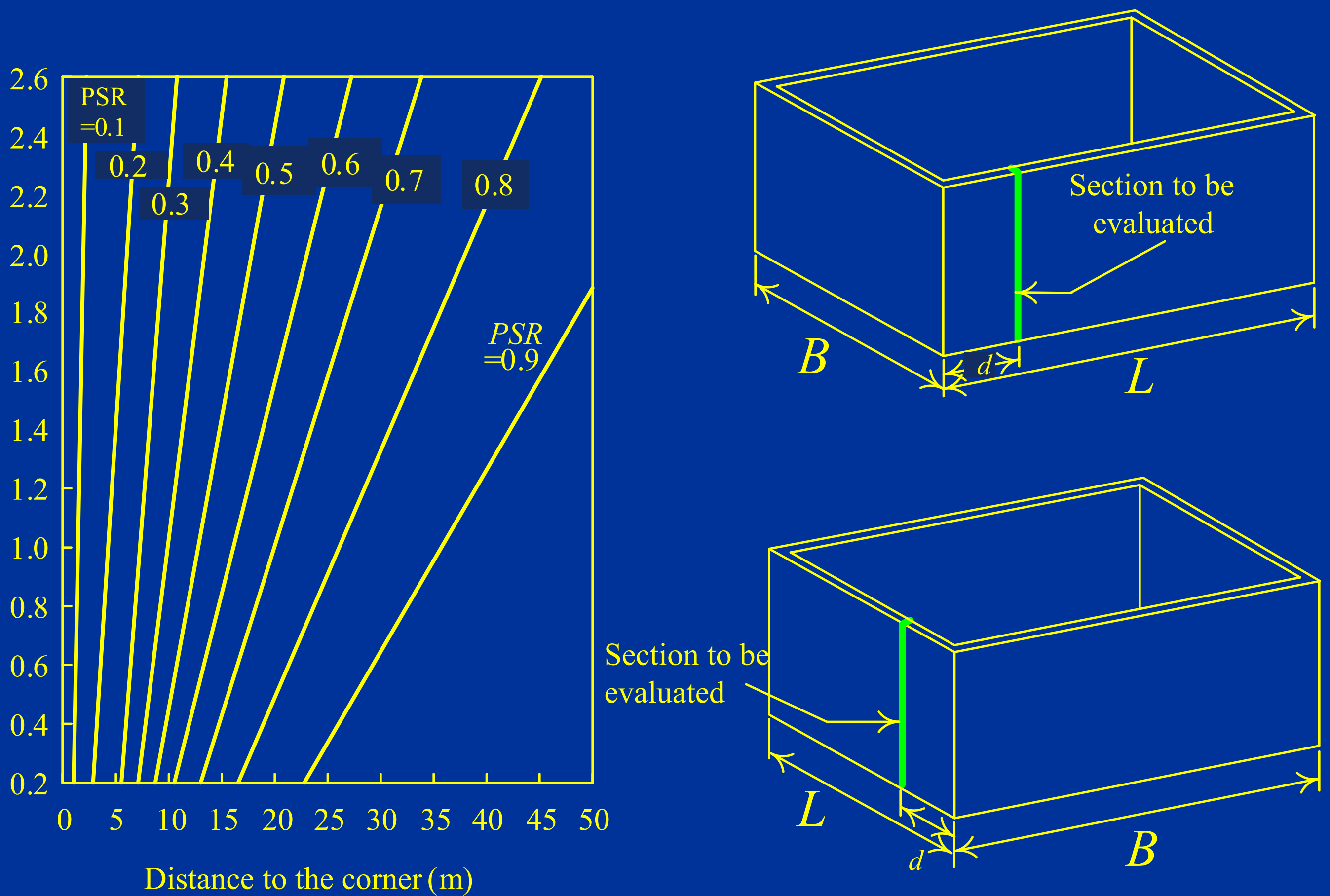


FIGURE 8.29 Comparisons of the wall deflection from plane strain analysis, three dimensional analysis and field measurement respectively in a corner of the Haihaw Financial Center excavation



B=Width *L*= Length *d*=Distance to the corner *PSR*=Plane strain ratio

(a) (b)

FIGURE 6.30 Relationship between the plane strain ratio and the aspect ratio of an excavation
(a) *PSR*, the length-width ratio, and the distance from the corner
(b) symbol explanation

8.11 Discussion of accuracy of analysis results

The bending moment (or stress) of a retaining wall is the multiplication of EI and the second derivative of the deformation curve.
Therefore, as long as the accurate deformation curve is obtained, the bending moment of the wall can also be obtained accurately.

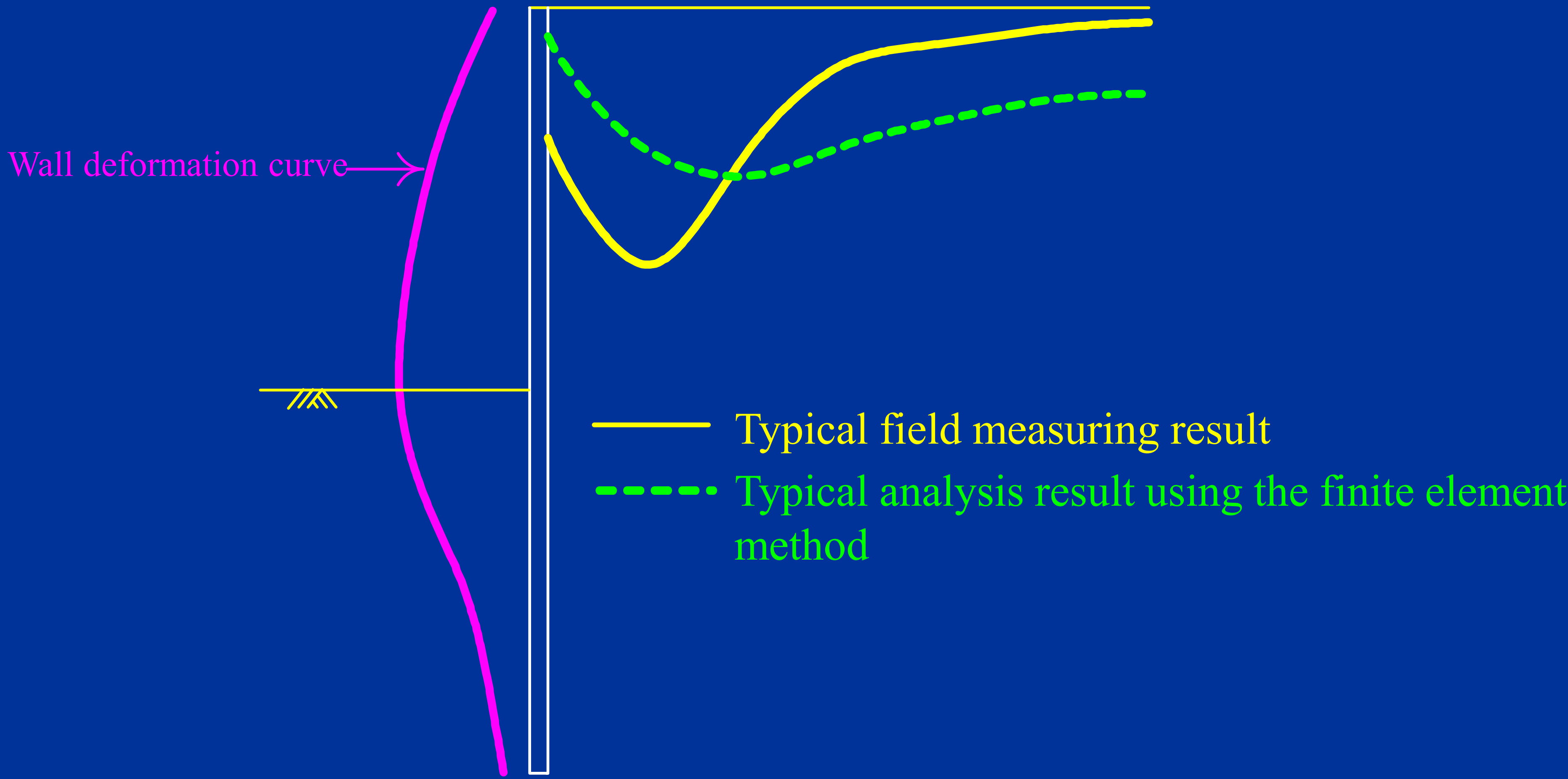


FIGURE 8.35 Comparison of excavation-induced settlement derived from a typical finite element analysis with field measurement

(2) Wang (1997):

Numerical program:

FLAC

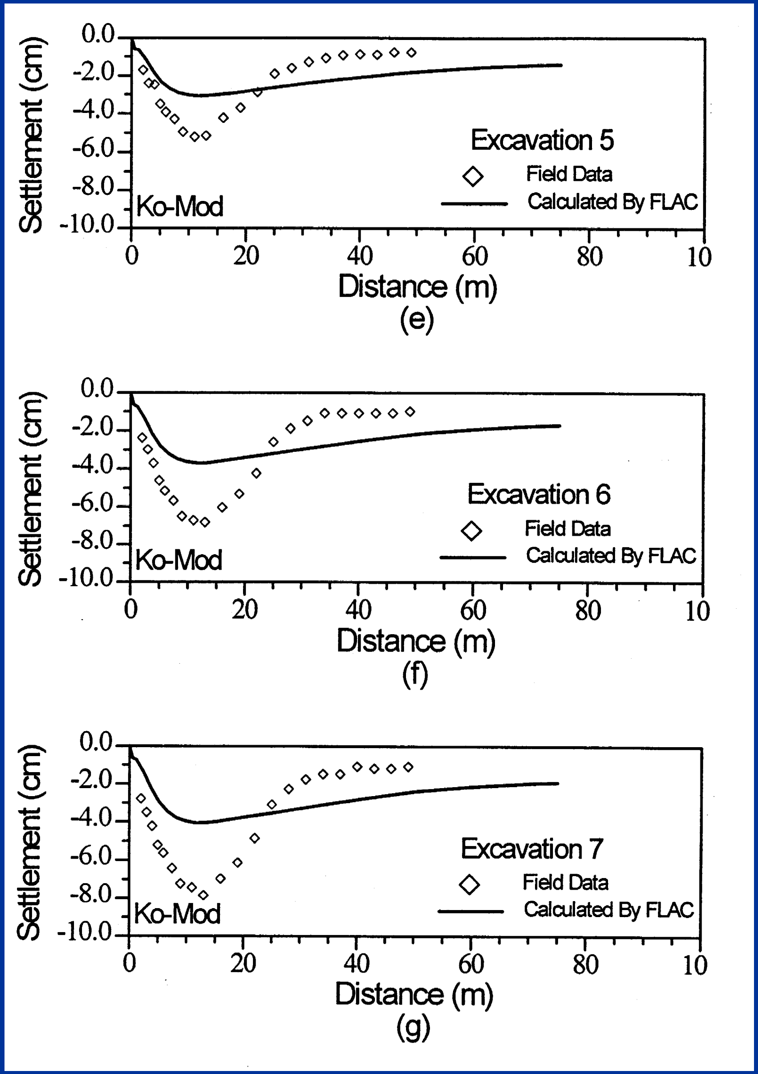
Soil model:

Creep model

hyperbolic model

Excavation case:

TNEC

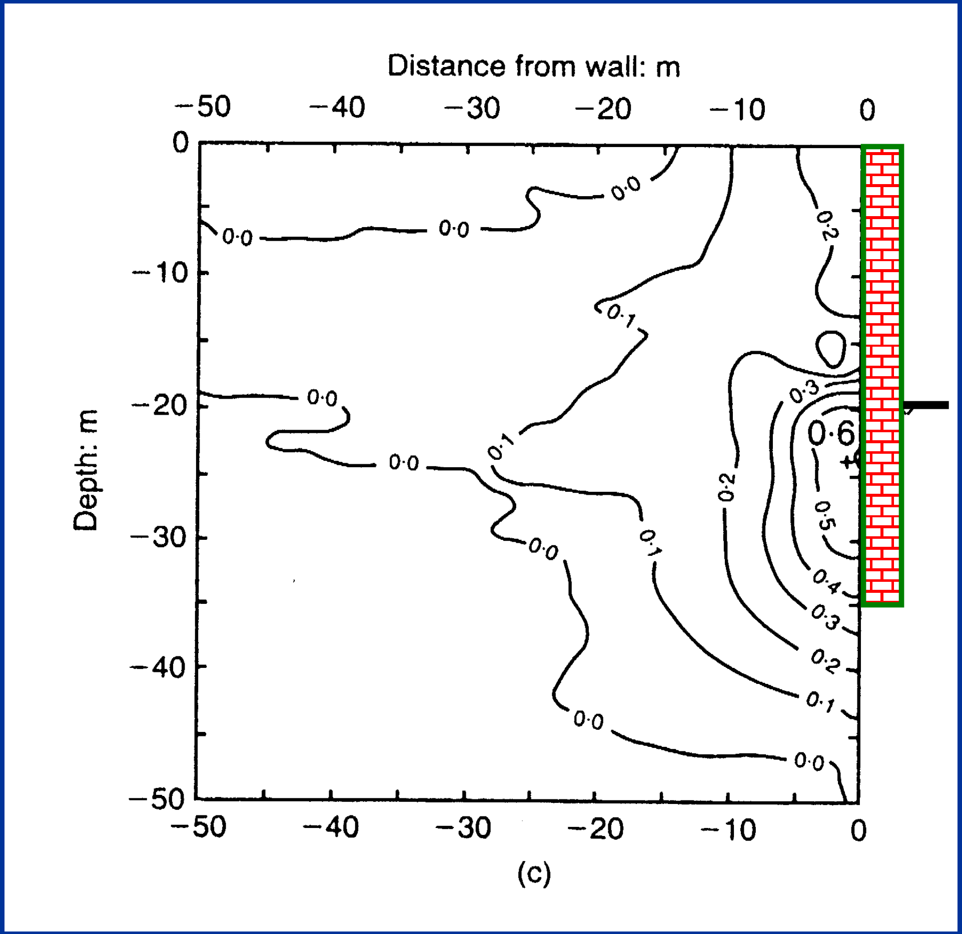


Ground shear strain behind the diaphragm wall induced by excavation at different stages

TNEC case

Strain contour

at stage 13 (unit: %)



Why can the surface settlement not be precisely simulated?

The reasons are

↓

Numerical program?

Soil model?

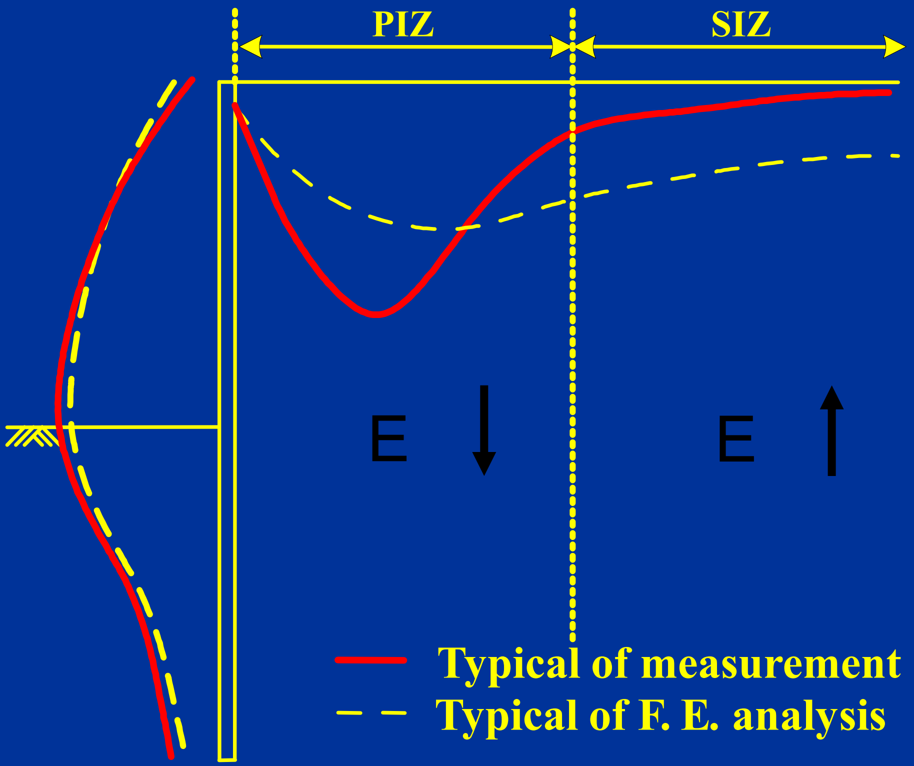
Parameters?

or

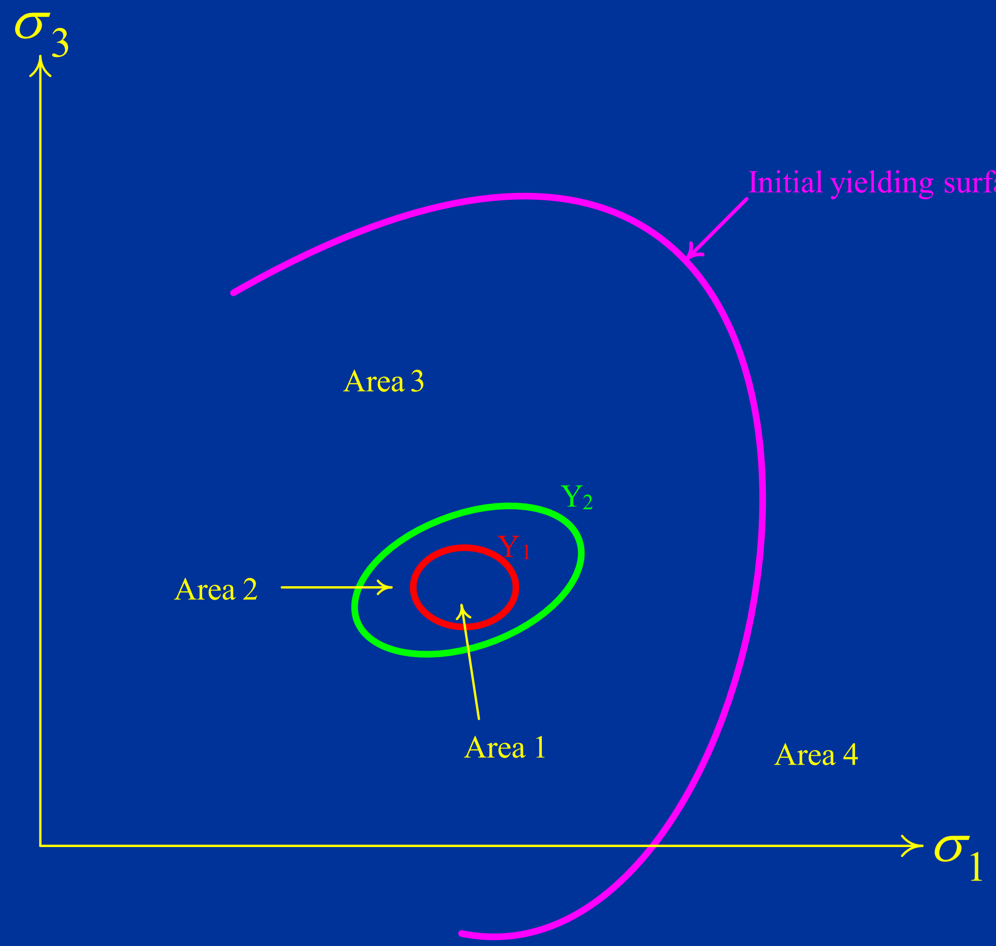
Method of simulation?

↓

The reasons may be attributed to the soil model, in which the certain soil behavior is not considered.

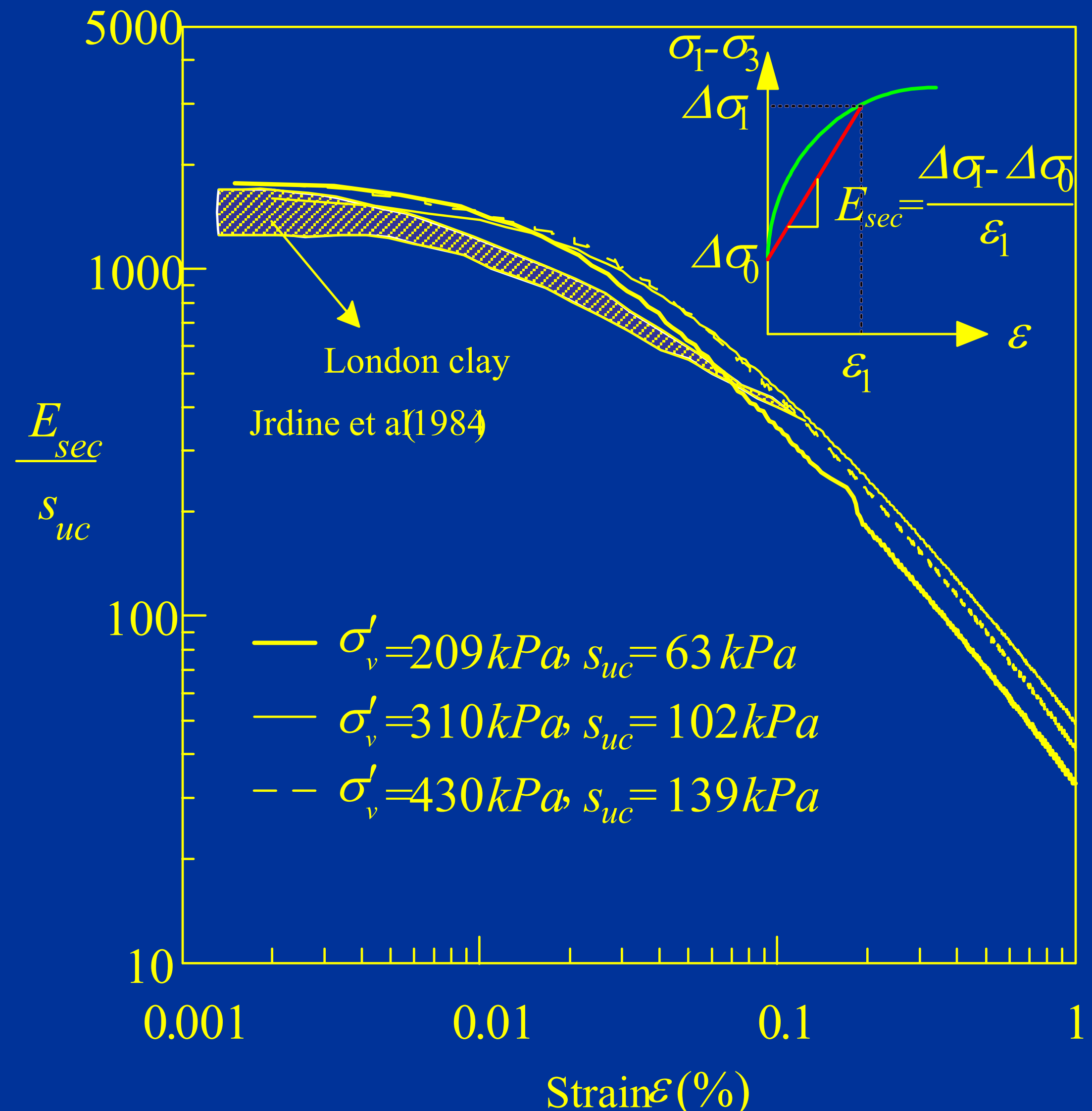


Yielding surfaces for clayey soils



Common material??

Variation of normalized secant modulus with strain



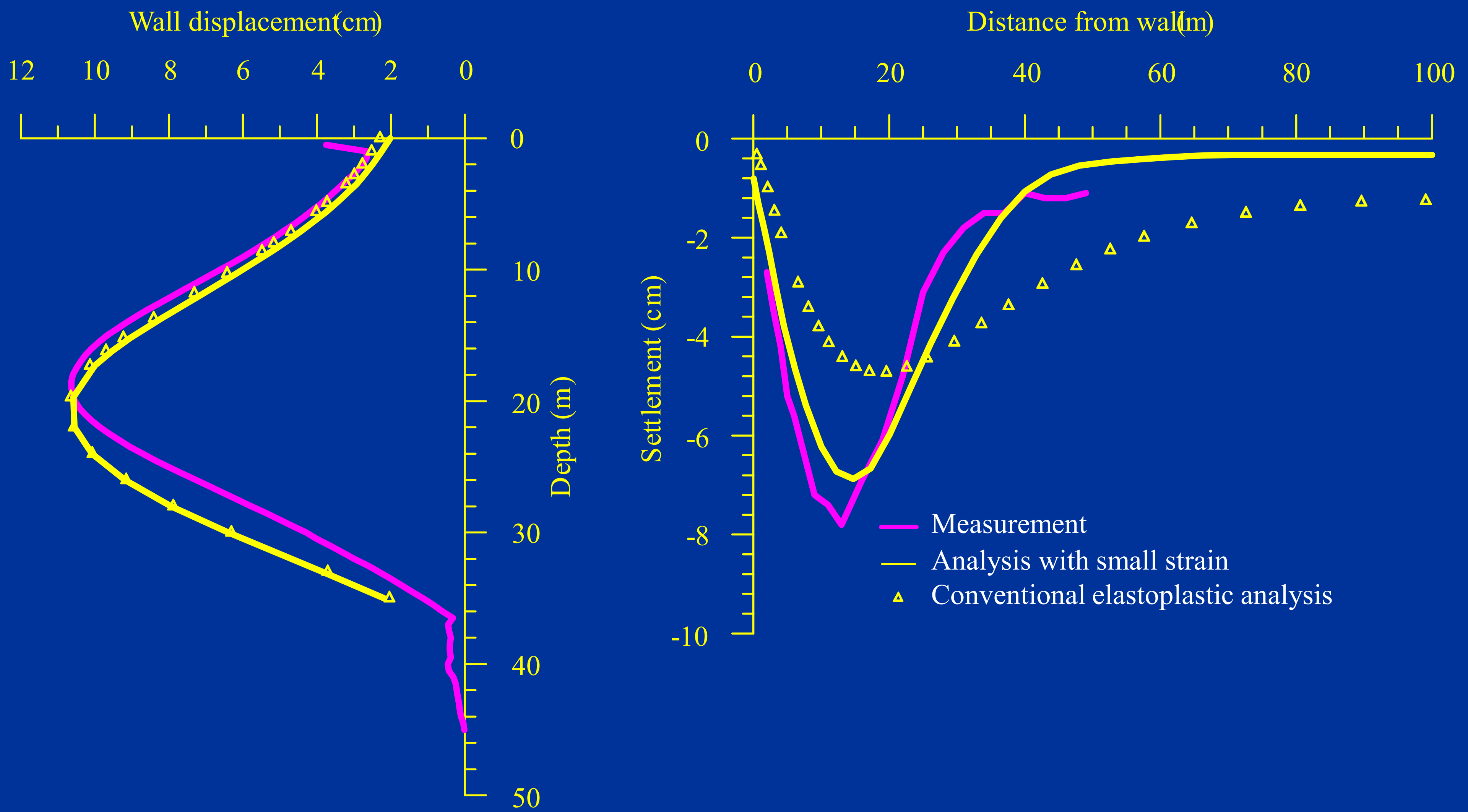


FIGURE8.37 Comparison between the measured and analytic wall displacements and surface settlements at the final stag

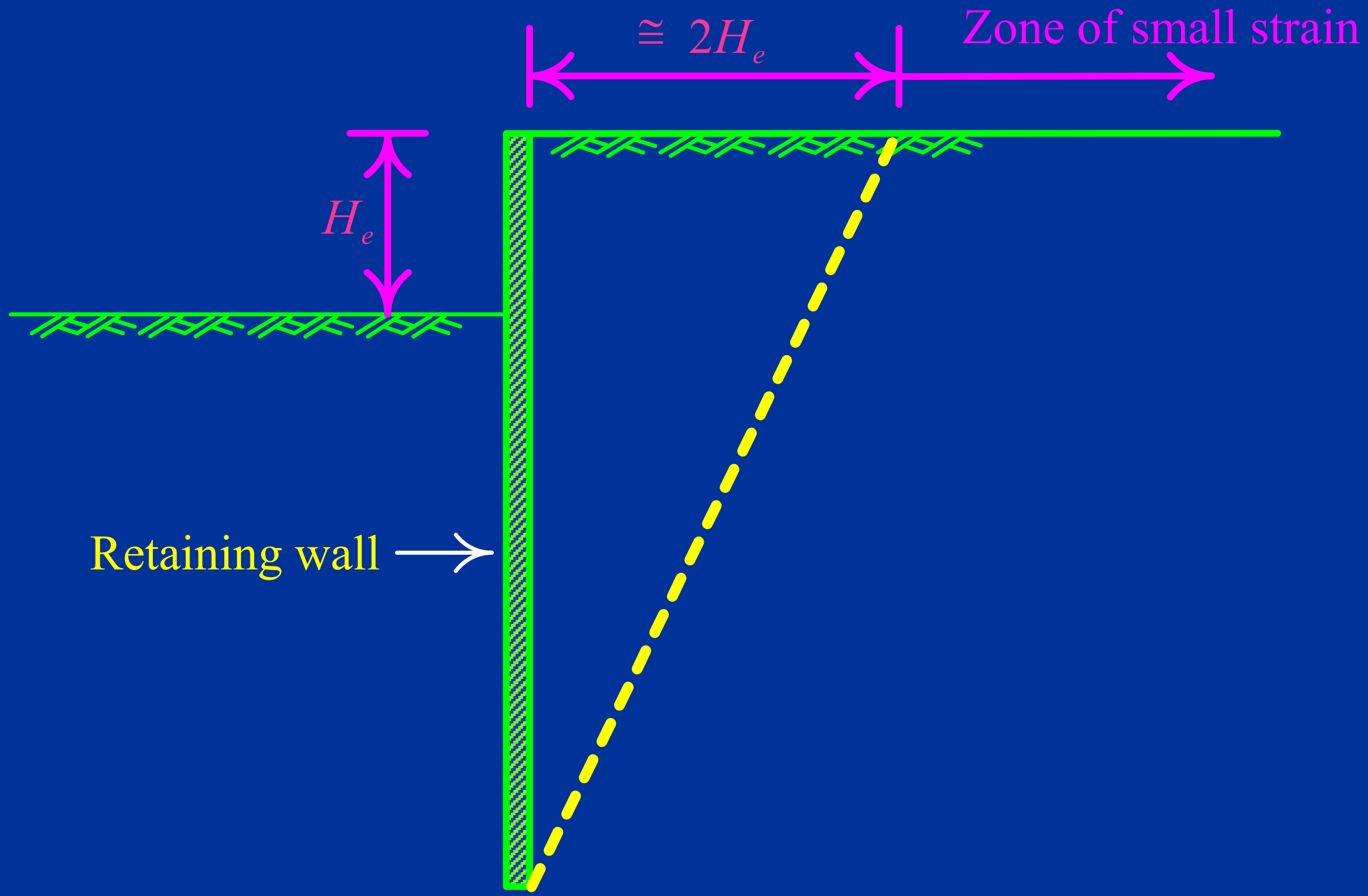


FIGURE 8.38 Zone of small strain in an excavation