B-24

KULIAH UMUM

PERENCANAAN UPPER & SUB STRUCTURE DALAM MODELISASI DAN KONSEP DISAIN YANG DIGUNAKAN SECARA PRAKTISI & AKADEMISI

Pemilihan Pengunaan Parameter Tanah (Soil Properties) Dalam Modelisasi Sub Structure Pada Analisis Mengunakan F.E.M

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28 November 2002

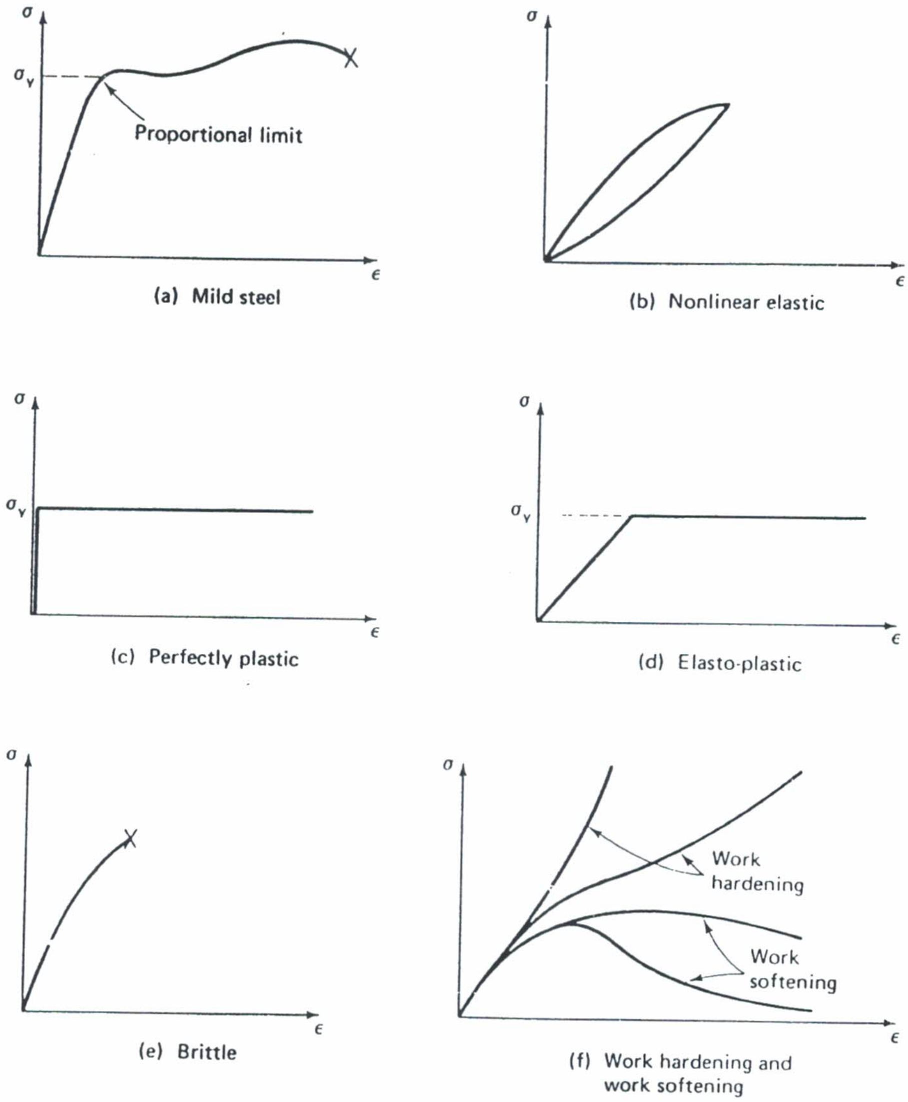


Fig. 10.4 Examples of stress-strain relationships for ideal and real materials: (a) mild steel, (b) nonlinear elastic, (c) perfectly plastic, (d) elasto-plastic, (e) brittle, and (f) work-hardening and work-softening.

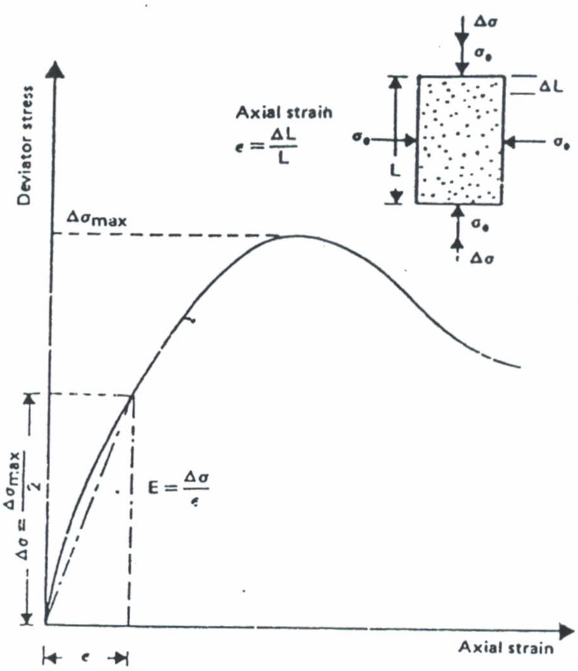


Fig. 6.12 Young's modulus from triaxial test.

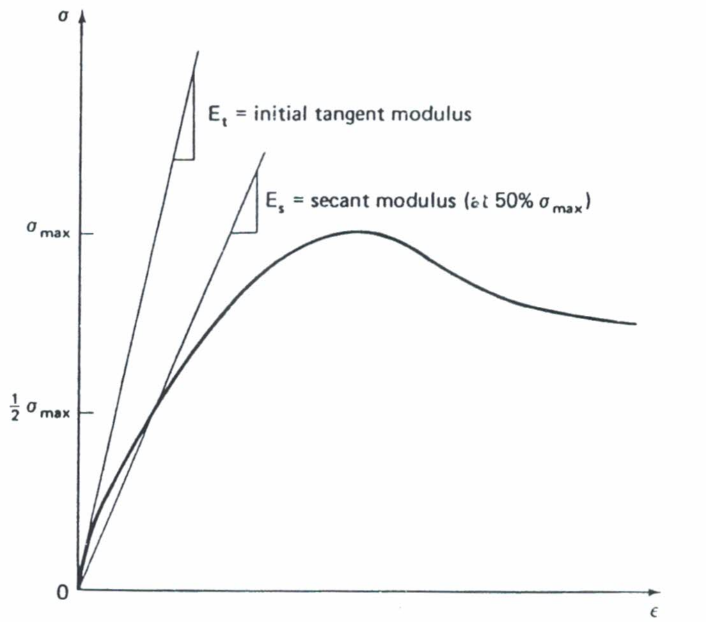


Fig. 11.61 Definitions of the initial tangent modulus and the secant modulus (usually defined at 50% of the maximum stress).

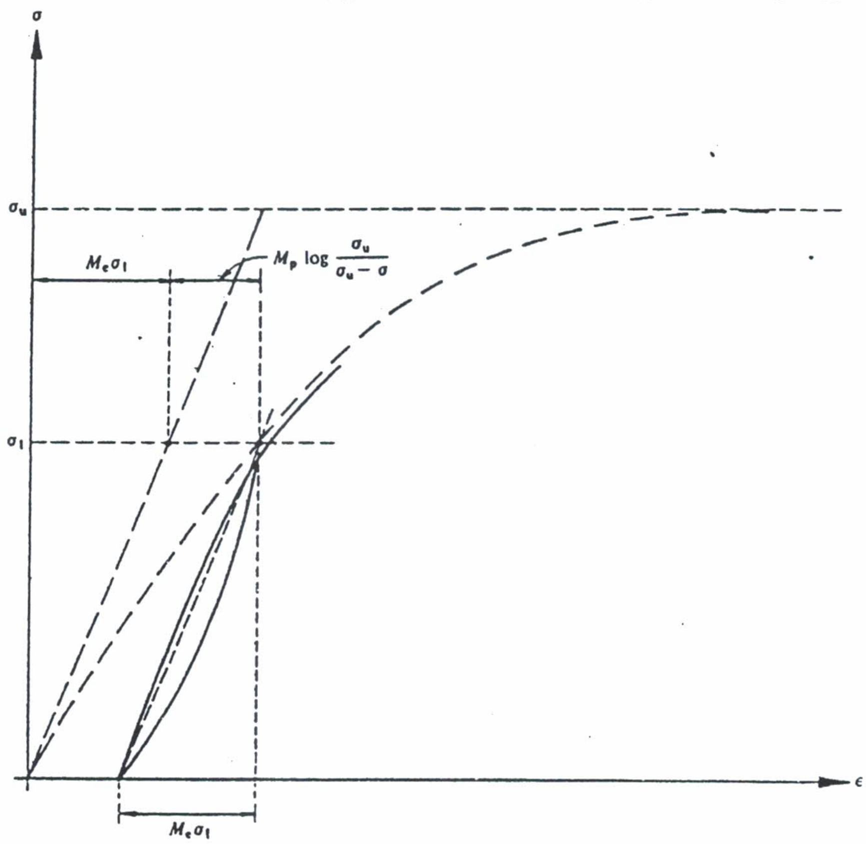


Fig. 10-II.3 Stress-strain curve.

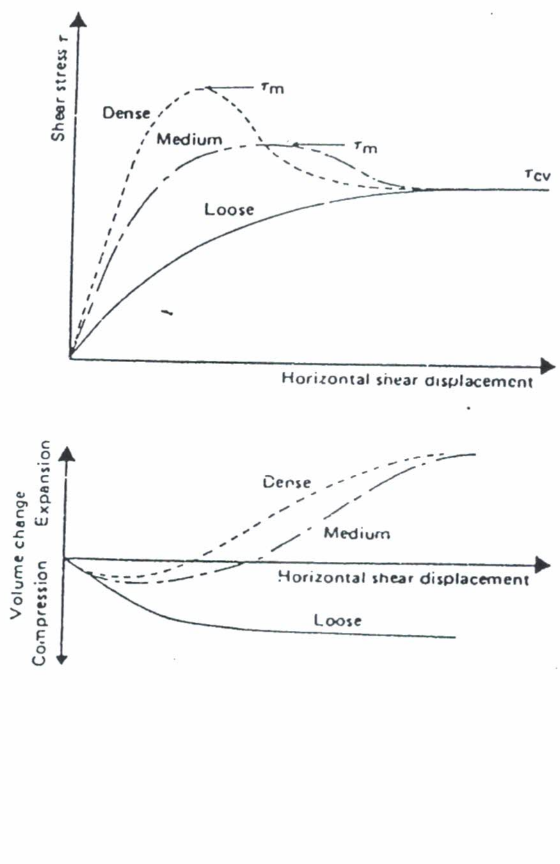


Fig. 7.3 Direct shear test results in loose, medium, and dense sands.

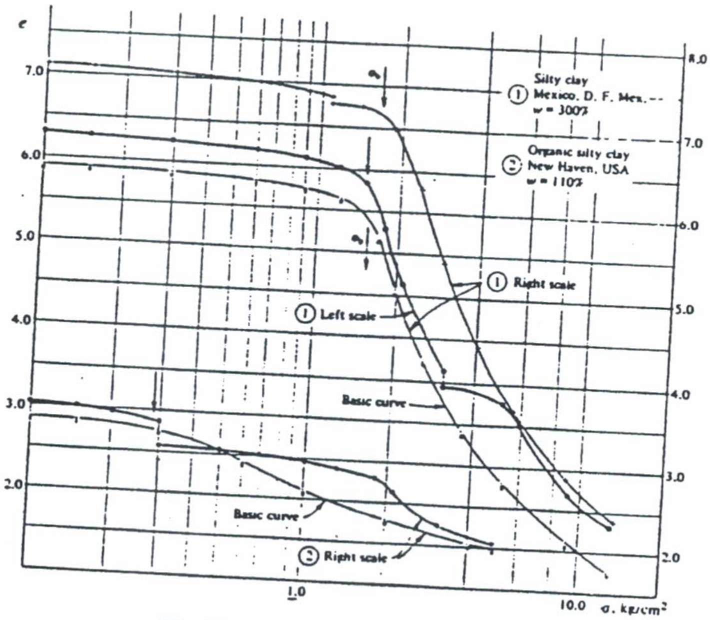
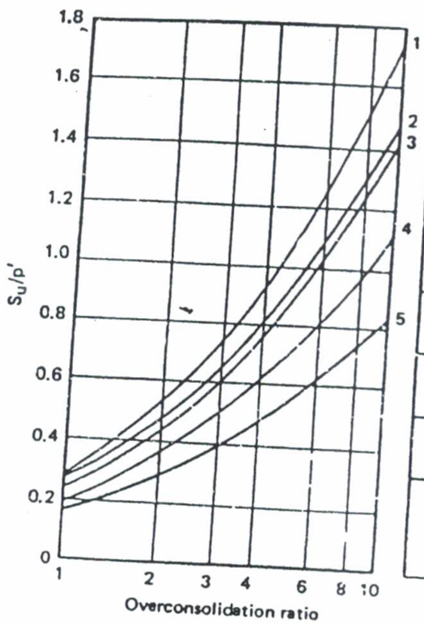


Fig. 18-II.3 Hardening tests of clay.

Compressibility	m_v cm^2/kg	ν	Sediment
Very high	greater than 0.1	0.43 to 0.35	Lacustrine clays and silts
High	0.1–0.02	0.35 to 0.30	Clays and silts, lacustrine sandy silts. Residual soils. Loose volcanic dust
Medium	0.02–0.005	0.30 to 0.25	Compact clays and silts, fine eolian sediments. Residual soils and volcanic semi-compact sediments. Fine alluvium
Low	0.005–0.002	0.25	Sand, compact silts, alluvial soils. Compact and well graded sediments
Very low	less than 0.002	0.25	Sands, gravelly soils. Compact alluvial sediments, cemented and well graded



Curve no.	Soil	W_n, %	PI
1	Maine organic clay	65	34
2	Bangkok clay	65	41
3	Atchafalaya clay	45	75
4	Boston blue clay	41	21
5	Connecticut Varve clay Clay layer Slit layer	 65 35	 39 12

Fig. 7.42 S_u/p' for several clays. *(Redrawn after C. C. Ladd and R. Foot, New Design Procedures for Stability of Soft Clays,* J. Geotech. Eng. Div., ASCE, *vol. 100, no. GT7, 1974.)*

SIFAT ELASTIS PADA TANAH

Modulus tegangan-regangan E_s, modul geser G', rasio Poisson μ, dan modulus reaksi bagian-bawah k_s adalah sifat-sifat elastik yang penting. Nilai-nilai ini umumnya dipakai untuk menghitung perkiraan-perkiraan penurunan pondasi. Modulus geser G' umumnya dipakai pada masalah getaran untuk memperkirakan amplitudo perpindahan dan frekuensi pondasi.

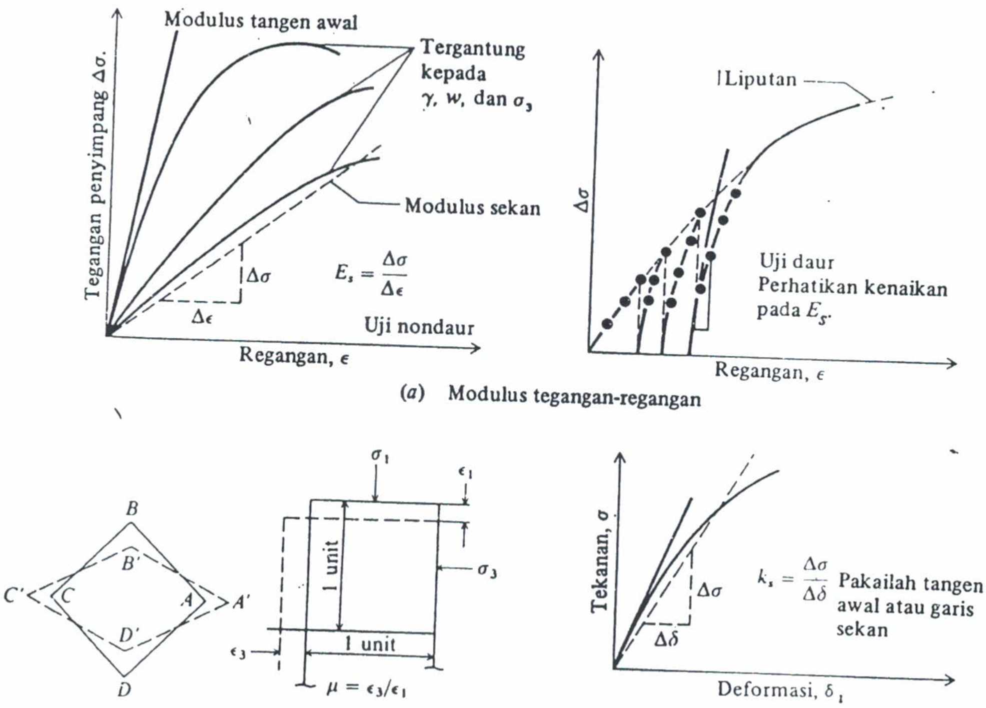
Modulus tegangan-regangan itu dapat diperoleh dari lereng (tangen dan sekan) kurva tegangan-regangan pada pengujian triaksial (lihat Gambar 2-37*a* dan 2-26). Sering diperkirakan dari uji lapangan pada Bab 3 (lihat juga Tabel 5-5). Rentang nilai yang khas untuk beberapa tanah diberikan pada Tabel 2-7. Dapat terlihat bahwa E_s untuk tanah itu hanya 1/10 sampai 1/100-nya dibandingkan dengan baja dan beton.

Rasio Poisson dipakai untuk mengkaji penurunan dan getaran. Hal itu ditentukan sebagai rasio kompresi poros terhadap regangan pemuaian lateral sebagai

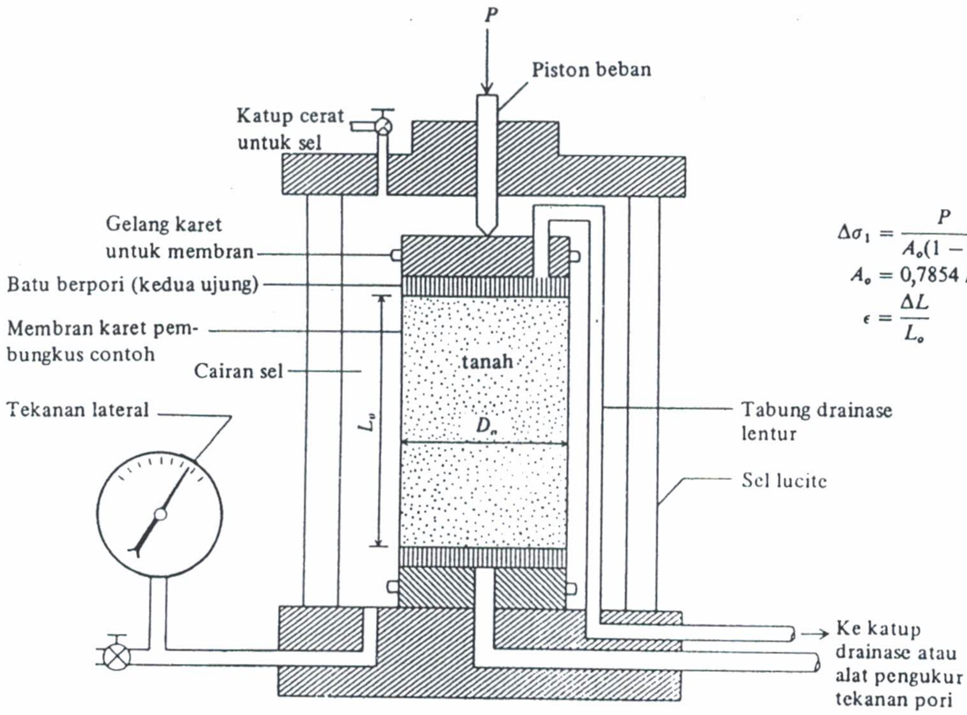
$$\mu = \frac{\epsilon_3}{\epsilon_1} \tag{a}$$

Pada kasus yang umum ada tiga nilai untuk tanah anisotropik dengan poros "1" yang pada gilirannya mengambil ketiga arah koordinat dan "3" poros yang tegaklurus terhadap masing-masing. Pada kebanyakan tanah, kondisi isotropik itu diasumsikan sehingga hanya ada satu nilai μ.

Dari definisi μ terbukti bahwa μ itu dapat negatif kalau terdapat pengkerutan lateral. Bukti eksperimental menunjukkan bahwa μ mungkin lebih besar dari 0,5 (yang menunjukkan pemuaian volume yang besar—atau paling tidak, terdapat regang lateral yang besar—selama pergeseran). Secara ketat, $\mu > 0,5$ menunjukkan keadaan tanah yang plastis di mana



GAMBAR 2-37 Sifat-sifat elastis pada tahan



$$\Delta\sigma_1 = \frac{P}{A_o(1-\epsilon)}$$
$$A_o = 0{,}7854\, D_o^2$$
$$\epsilon = \frac{\Delta L}{L_o}$$

GAMBAR 2-23 Rincian garis utama untuk sel triaksial.

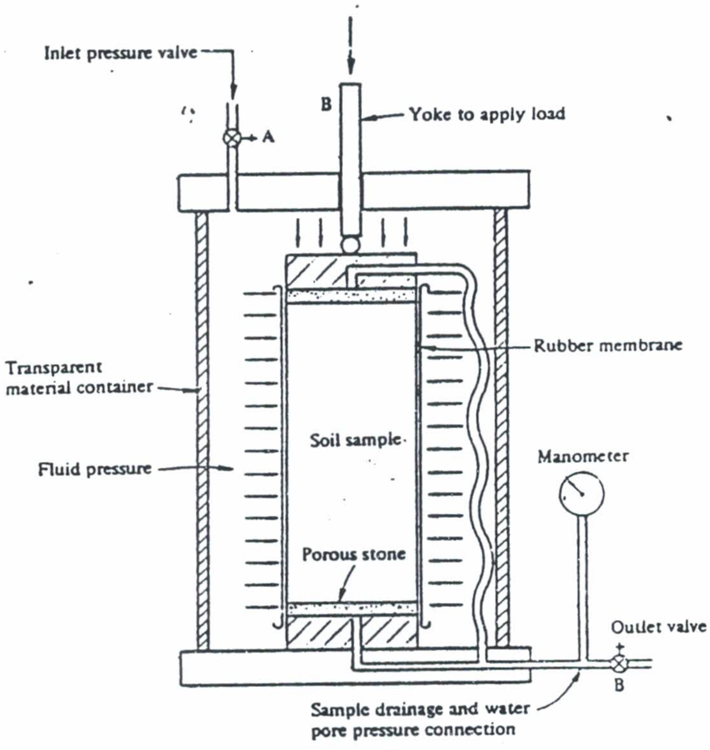
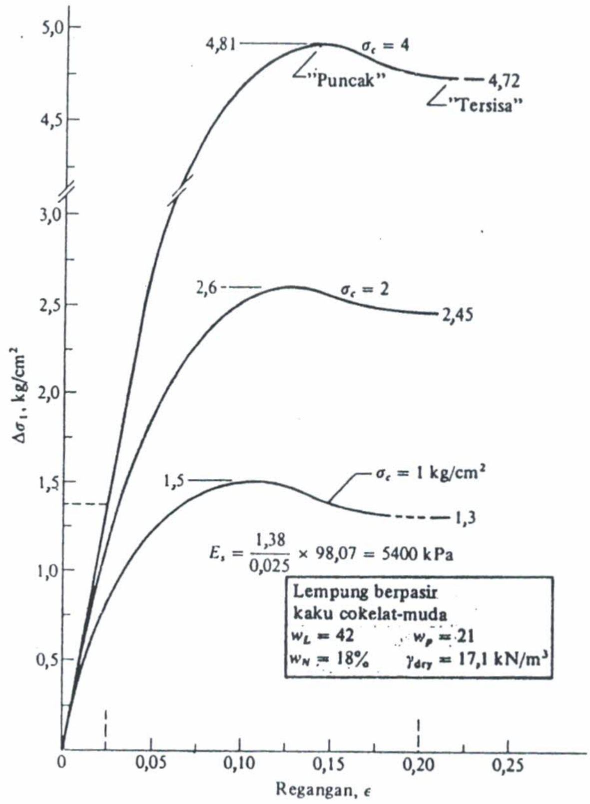


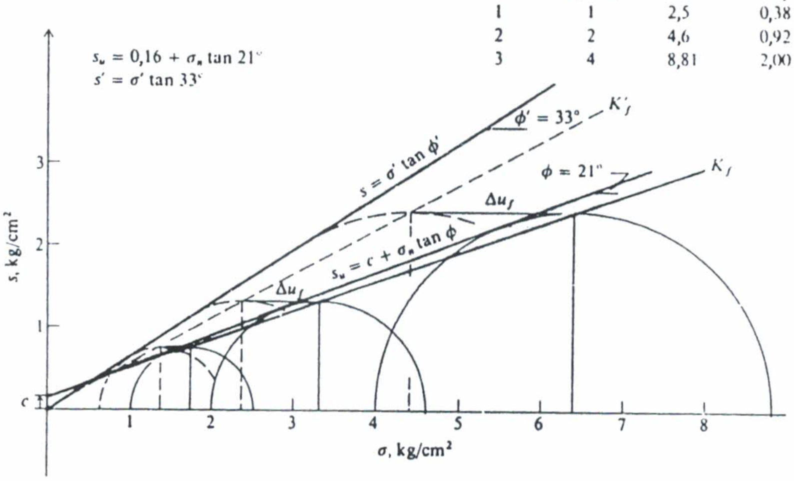
Fig. 25-11.3 Triaxial compression chamber.



(*a*) Data tegangan-regangan dengan modulus tegangan-regangan E_s menurut penghitungan.

Data uji

No.	$\sigma_3 = \sigma_c$	σ_1	Δu_f
1	1	2,5	0,38
2	2	4,6	0,92
3	4	8,81	2,00



(*b*) Lingkaran-lingkaran Mohr dirajahkan untuk tegangan total maupun tegangan efektif dari data yang dinyatakan pada bagian *a*.

GAMBAR 2-26 Uji triaksial CU dengan tekanan pori diukur untuk suatu tanah kohesif terkonsolidasi normal.

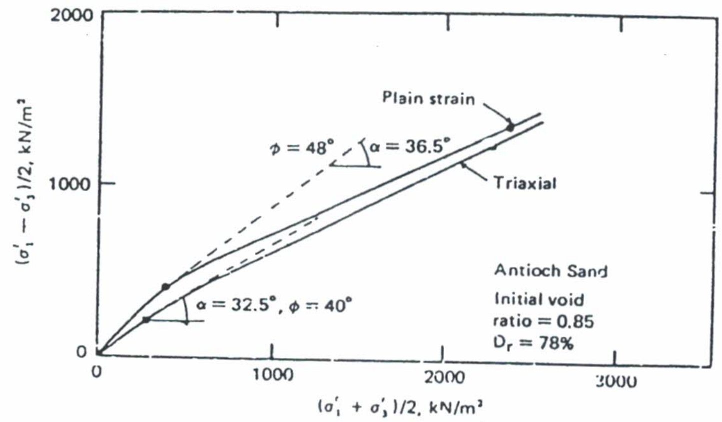


Fig. 7.16 Strength of Antioch sand under drained condition. *(Redrawn after K. L. Lee, Comparison of Plane Strain and Triaxial Tests on Sand,* J. Soil Mech. Found. Div., ASCE, *vol. 96, no. SM3, 1970.)*

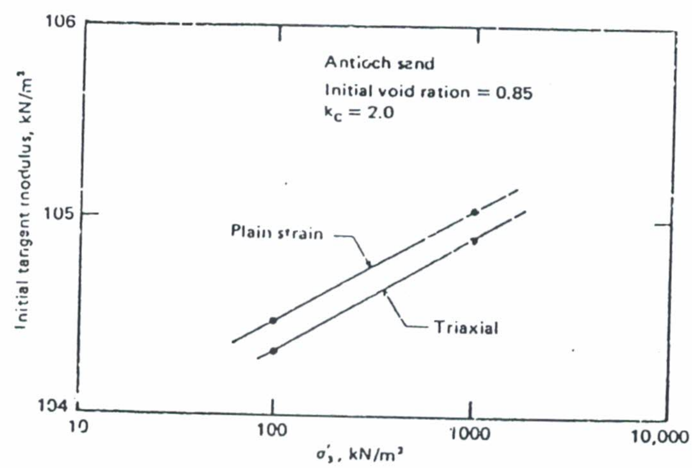


Fig. 7.17 Initial tangent modulus from drained tests on Antioch sand. *(Redrawn after K. L. Lee, Comparison of Plane Strain and Triaxial Tests on Sand,* J. Soil Mech. Found. Div., ASCE, *vol. 96, no. SM3, 1970.)*

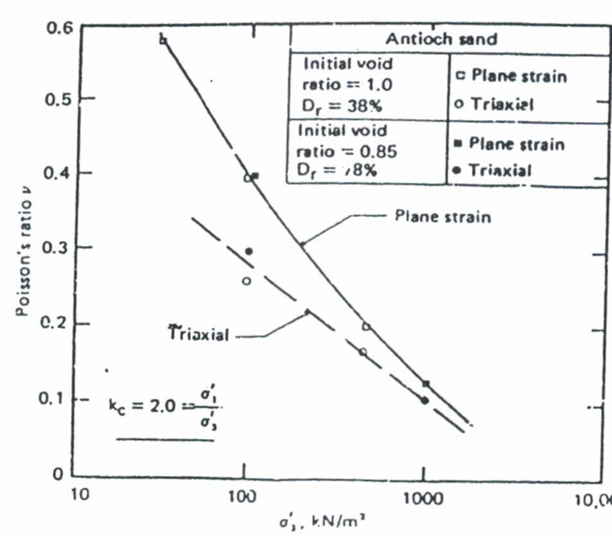


Fig. 7.18 Poisson's ratio from drained tests on Antioch sand. *(Redrawn after K. L. Lee, Comparison of Plane Strain and Triaxial Tests on Sand,* J. Soil Mech. Found. Div., ASCE, *vol. 96, no. SM3, 1970.)*

untuk beberapa jenis tanah

Nilai lapangan bergantung pada sejarah tegangan, kadar air, kerapatan, dan lain sebagainya.

Tanah	E_s Ksf	E_s MPa
Lempung		
Sangat lunak	50–250	2–15
Lunak	100–500	5–25
Sedang	300–1000	15–50
Keras	1000–2000	50–100
Berpasir	500–5000	25–250
Laci es		
Lepas	200–3200	10–153
Padat	3000–15000	144–720
Sangat padat	10000–30000	478–1440
Tanah lus (loss)	300–1200	15–60
Pasir		
Berlanau	150–450	5–20
Lepas	200–500	10–25
Padat	1000–1700	50–81
Pasir dan kerikil		
Lepas	1000–3000	50–150
Padat	2000–4000	100–200
Serpih	3000–300000	150–5000
Lanau	40–400	2–20

150 – 500

Persamaan-persamaan untuk Modulus Regangan-Tegangan E_s dengan beberapa metode percobaan.

E_s di dalam kPa untuk SPT dan satuan-satuan q_c untuk CPT; bagilah kPa dengan 50 untuk mendapatkan ksf. Nilai-nilai N harus diperkirakan sebagai N_{55} dan bukan N_{70}

Tanah	SPT	CPT
Pasir (terkonsolidasi normal)	$E_s = 500(N + 15)$ $E_s = (15\,000 \text{ to } 22\,000) \ln N$ $E_s§ = (35\,000 \text{ to } 50\,000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s† = (1 + D_r^2)q_c$
Pasir (jenuh)	$E_s = 250(N + 15)$	
Pasir (terkonsolidasi-lebih)	$E_s‡ = 18\,000 + 750N$ $E_{s(OCR)} = E_{s(nc)}\,(OCR)^{1/2}$	$E_s = 6 \text{ to } 30q_c$
Pasir berkerikil dan kerikil	$E_s = 1200(N + 6)$ $E_s = 600(N + 6) \quad N \leq 15$ $E_s = 600(N + 6) + 2000 \quad N > 15$	
Pasir berlempung	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Pasir berlanau	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Lempung lunak	—	$E_s = 3 \text{ to } 8q_c$
Lempung	Memakai kekuatan geser tak tersalur S_u dalam satuan S_u $I_P > 30$ atau organik $I_P < 30$ atau kaku $E_{s(OCR)} = E_{s(nc)}\,(OCR)^{1/2}$	 $E_s = 100 \text{ to } 500s_u$ $E_s = 500 \text{ to } 1500s_u$

† Vesic (1970.

‡ D'Appolonia bersama kawan-kawan (1970) dari persamaan pengarang.

§ USSR (dan bukan standar perhitungan pukulan N)

Sumber umum: European Conference on Standard Penetration. Testing (1974), vol. 2.1, pp 150–151; CGJ, November 1983, pp. 726–737;. Use of In Situ Tests in Geotechnical Engineering, ASCE (1986),p.

Jangkauan nilai banding Poisson μ

Jenis tanah	μ
Lempung jenuh	0,4–0,5
Lempung tak jenuh	0,1–0,3
Lempung berpasir	0,2–0,3
Lanau	0,3–0,35
Pasir (padat) pasir berkerikil	0,1–1,00
biasa dipakai	0,3–0,4
Batuan (rock)	0,1–0,4 (agak bergantung pada jenis batuan)
Tanah lus	0,1–0,3
Es	0,36
Beton	0,15

Table 6.5 Recommended values of E and ν

Type of Soil	Properties of soil*	Void ratio e: 0.41 to 0.5	0.51 to 0.6	0.61 to 0.70
Sand (coarse)	ϕ	43	40	38
$\nu = 0.15$	E (lb/in^2)	6,550	5,700	4,700
	E (kN/m^2)	45,200	39,300	32,400
Sand (medium	ϕ	40	38	35
coarse)	E (lb/in^2)	6,550	5,700	4,700
$\nu = 0.2$	E (kN/m^2)	45,200	39,300	32,400
Sand (fine	ϕ	38	36	32
grained)	E (lb/in^2)	5,300	4,000	3,400
$\nu = 0.25$	E (kN/m^2)	36,600	27,600	23,500
Sandy silt	ϕ	36	34	30
$\nu = 0.3$ to 0.35	E (lb/in^2)	2,000	1,700	1,450
	E (kN/m^2)	13,800	11,700	10,000

From *Foundations of Theoretical Soil Mechanics* by M. E. Harr. Copyright © 1966 McGraw-Hill Book Company, New York. Used with the permission of McGraw-Hill Book Company.

*Conversion factor: 1 lb/in^2 = 6.9 kN/m^2 (the values of kN/m^2 have been rounded off). ϕ is the drained friction angle.

Table 6.6 Young's modulus for vertical static compression of sand from standard penetration number

Reference	Relationship*	Soil types	Basis	Remarks
Schultze and Meizer (1965)	$E_s = \nu\sigma_o^{0.522}$ kg/cm^2 $\nu = 246.2 \log N - 263.4\, \sigma_o + 375.6 \pm 57.6$ $0 < \sigma_o < 1.2$ kg/cm^2 σ_o = effective overburden pressure	Dry sand	Penetration tests in field and in test shaft. Compressibility based on e, e_{max}, and e_{min} (Schultze and Moussa, (1961)	Correlation coefficient = 0.730 for 77 tests
Webb (1969)	$E_s = 5(N + 15)$ ton/ft^2 $E_s = 10/3(N + 5)$ ton/ft^2	Sand Clayey sand	Screw plate tests	Below water table
Farrent (1963)	$E_s = 7.5(1 - \nu^2)N$ ton/ft^2 ν = Poisson's ratio	Sand	Terzaghi and Peck loading settlement curves	
Begemann (1974)	$E_s = 40 + C(N - 6)$ kg/cm^2 $N > 15$ $E_s = C(N + 6)$ kg/cm^2 $N < 15$ $C = 3$ (silt with sand) to 12 (gravel with sand)	Silt with sand to gravel with sand		Used in Greece
Trofimenkov (1974)	E_s = (350 to 500) $\log N$ kg/cm^2	Sand		U.S.S.R. practice

After J. K. Mitchell and W. S. Gardner, In Situ Measurement of Volume Characteristic, *Proc. Specialty Conference of the Geotechnical Engineering Division, ASCE*, vol. 2, 1975.

*N = standard penetration number. Note: 1 kgf/cm^2 = 98.1 kN/m^2; 1 ton/ft^2 = 95.6 kN/m^2.

Table 6.7 Equivalent Young's modulus for vertical static compression of sand–static cone resistance

Reference	Relationship	Soil types	Remarks
Buisman (1940)	$E_s = 1.5q_c$	Sands	Overpredicts settlements by a factor of about 2
Trofimenkov (1964)	$E_s = 2.5q_c$ $E_s = 100 + 5q_c$	Sand	Lower limit Average
De Beer (1967)	$E_s = 1.5q_c$	Sand	Overpredicts settlements by a factor of 2
Schultze and Melzer (1965)	$E_s = \frac{1}{m_v} \nu \sigma_o^{0.522}$ $\nu = 301.1 \log q_c - 382.3\sigma_o \pm 60.3 \pm 50.3$ σ_o = effective overburden pressure	Dry sand	Based on field and lab penetration tests—compressibility based on e, e_{max}, and e_{min} Correlation coefficient = 0.778 for 90 tests valid for σ_o = 0 to 0.8 kg/cm²
Bachelier and Parez (1965)	$E_s = \alpha q_c$ α = 0.8 to 0.9 α = 1.3 to 1.9 α = 3.8 to 5.7 α = 7.7	 Pure sand Silty sand Clayey sand Soft clay	
Thomas (1968)	$E_s = \alpha q_c$ α = 3 to 12	3 sands	Based on penetration and compression tests in large chambers Lower values of α at higher values of q_c; attributed to grain crushing
Webb (1969)	$E_s = \frac{5}{2}(q_c + 30)$ ton/ft² $E_s = \frac{5}{3}(q_c + 15)$ lb/ft²	Sand below water table Clayey sand below water table	Based on screw plate tests: correlated well with settlement of oil tanks
Vesic (1970)	$E_s = 2(1 + D_R^2)q_c$ D_r = relative density	Sand	Based on pile load tests and assumptions concerning state of stress
Schmertmann (1970)	$E_s = 2q_c$	Sand	Based on screw plate tests
Bogdanović (1973)	$E_s = \alpha q_c$ $q_c > 40$ kg/cm², $\alpha = 1.5$ $20 < q_c < 40$, α = 1.5 to 1.8 $10 < q_c < 20$, α = 1.8 to 2.5 $5 < q_c < 10$, α = 2.5 to 3.0	 Sand, sandy gravels Silty saturated sands Clayey silts with silty sand and silty saturated sands with silt	Based on analysis of soil settlements over a period of 10 years
Schmertmann (1974)	$E_s = 2.5q_c$ $E_s = 3.5q_c$	NC sands NC sands	L/B = 1 to 2, axisymmetric $L/B > 10$, plane strain
De Beer (1974)	$E_s = 1.6q_c - 8$	Sand	Bulgarian practice
	$E_s = 1.5q_c$, $q_c > 30$ kg/cm² $E_s = 3q_c$, $q_c < 30$ kg/cm²	Sand	Greek practice
	$E_s > 1.5q_c$ or $E_s = 2q_c$	Sand	Italian practice
	$E_s = 1.9q_c$ $E_s = \frac{5}{2}(q_c + 3200)$ kN/m² $E_s = \frac{5}{3}(q_c + 1600)$ kN/m²	Sand Fine to medium sand Clayey sands, $PI < 15\%$	South African practice
	$E_s = \alpha q_c$, $1.5 < \alpha < 2$	Sands	U.K. practice
Trofimenkov (1974)	$E_s = 3q_c$ $E_s = 7q_c$	Sands Clays	U.S.S.R. practice

After J. K. Mitchell and W. S. Gardner, in Situ Measurement of Volume Characteristics, *Proc. Specialty Conference of the Geotechnical Engineering Division, ASCE*, vol. 2, 1975.

Note: 1 kg/cm² = 98.1 kN/m²; 1 ton/ft² = 95.6 kN/m²; 1 lb/ft² = 47.8 N/m².

Table 6.8 Values of β from various case studies of immediate settlement

No.	Location of structure	Clay properties: Plasticity index	Sensitivity	Over-consolidation ratio	E_{field}, ton/m²	β	Source of S_u*
1	Oslo: Nine-story building	15	2	3.5	7,600	1,200	CIU
2	Asrum I: Circular load	16	100	2.5	990	1,000	Field vane
						1,200	CIU
3	Asrum II: Circular load test	14	100	1.7	880	1.000	Field vane
						1,100	CIU
4	Mastemyr: Circular load test	14	–	1.5	1,300	1,200	Field vane
						1,700	Bearing capacity
5	Portsmouth: Highway embankment	15	10	1.3	3,000	2,000	Field vane
						1,700	Bearing capacity
6	Boston: Highway embankment	24	5	1.5	10,000	1,600	Field vane
						1,200	CK_oU
				1.0	13,000	2,500	Field vane
						1,500	CK_oU
7	Drammen: Circular load test	28	10	1.4	3,200	1,400	Field vane
						1,100	CK_oU
8	Kawasaki: Circular load test	38	6 ± 3	1.0	2,200	400	Field vane
							CIU
9	Venezuela: Oil tanks	37	8 ± 2	1.0	5,00	800	CIU
10	Maine: Rectangular load test†	33 ± 2	4	1.5 to 4.5	100 to 200	80 to 160	UU and Bearing capacity

After D. J. D'Appolonia, H. G. Poulos, and C. C. Ladd, Initial Settlement of Structures on Clay, *J. Soil Mech. Found. Div., ASCF* vol. 97, no. SM10, 1971.

*Average value at a depth equal to the width of foundation. CIU = isotropically consolidated undrained shear test; UU = consolidated undrained shear test; CK_oU = consolidated undrained shear test with sample consolidated in K_o condition.

†Slightly organic plastic clay.

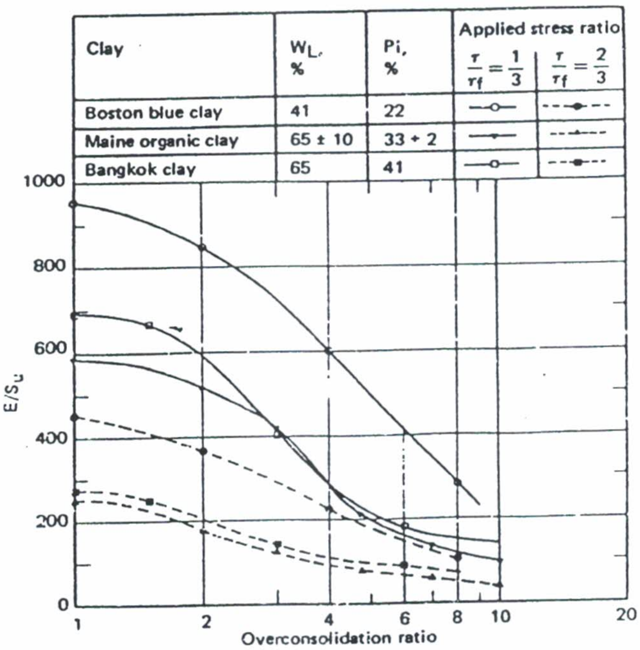


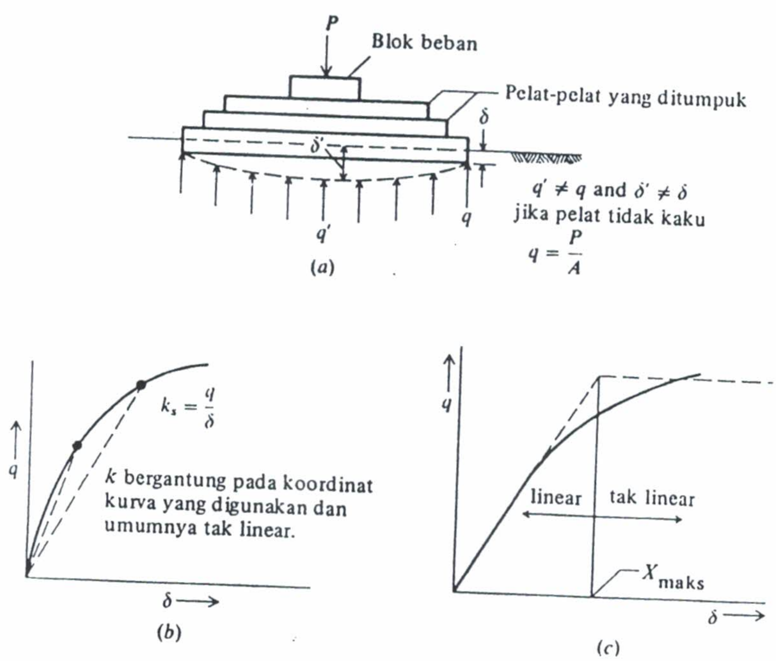
Fig. 6.14 Relationship between E/S_u and over consolidation ratio from CU tests on three clays determined from CK_0U-type direct shear tests. *(After D. J. D'Appolonia, H. G. Poulos, and C. C. Ladd, Initial Settlement of Structures on Clay,* J. Soil Mech. Found. Div., ASCE, *vol. 97, no. SM10, 1971.)*

FOUNDATION MODULUS

The contact stresses between the foundation slab and soil should be such that the distribution of stress is compatible with the deflections of the foundation structure and soil displacements, since, when loading the soil, the foundation structure and ground surface should undergo the same vertical displacements. Therefore, depending on the stress-strain-time characteristics of the soil, the contact stresses will change under the foundation slab. The subgrade reactions are determined using the concept of the unit foundation modulus defined by:

$$k = \frac{q}{\delta}$$

Penentuan modulus reaksi tanah dasar k.



Suatu persoalan utama adalah untuk memperkirakan nilai numerik k_s. Salah satu dari kontribusi awal adalah dari Terzaghi (1955), yang mengusulkan bahwa k_s untuk pondasi telapak berukuran penuh dapat diperoleh dari pengujian-pengujian beban pelat dengan menggunakan persamaan-persamaan yang berikut:

Untuk telapak di atas lempung:

$$k_s = k_1 B \tag{9-3}$$

Untuk pondasi telapak di atas pasir (termasuk efek-efek ukuran):

$$k_s = k_1 \left(\frac{B+1}{2B}\right)^2 \tag{9-4}$$

Untuk sebuah pondasi telapak empat persegi panjang di atas pasir yang berdimensi $B \times mB$

$$k_s = k_1 \frac{m + 0{,}5}{1{,}5\ m} \tag{9-5}$$

Di dalam persamaan-persamaan ini k_s = nilai yang diinginkan untuk telapak berukuran penuh dan k_1 = nilai dari sebuah pengujian beban pelat persegi sama sisi 1×1 kaki.

Vesic (1961*a*, 1961*b*) mengusulkan bahwa modulus reaksi tanah dasar dapat dihitung dengan menggunakan modulus tegangan-regangan E_s sebagai

$$k'_s = 0{,}65 \sqrt[12]{\frac{E_s B^4}{E_f I_f}} \frac{E_s}{1-\mu^2} \tag{9-6}$$

di mana E_s, E_f = modulus tanah dan telapak, secara berturut-turut, dalam satuan yang konsisten

B, I_f = lebar telapak dan momen inersia didasarkan pada penampang lintang (tidak terencana) dalam satuan yang konsisten

Salah satunya diperoleh k_s dari k'_s seperti

$$k_s = \frac{k'_s}{B}$$

Karena akar duabelas dari setiap nilai × 0,65 akan tertutup hingga 1, untuk memudahkan penggunaan Vesic' direduksi pada

Jangkauan Nilai-nilai Modulus Reaksi Tanah Dasar k_s.

Gunakan nilai-nilai sebagai panduan dan untuk perbandingan bila menggunakan persamaan-persamaan pendekatan.

Tanah	k_s, kcf	k_s, kN/m^3
Pasir lepas	30 – 100	4800 – 16000
Pasir padat sedang	60 – 500	9600 – 80000
Pasir padat	400 – 800	64000 – 128000
Pasir padat sedang berlempung	200 – 500	32000 – 80000
Pasir padat sedang berlanau	150 – 300	24000 – 48000
Tanah berlempung		
$q_u < 200$ k Pa (4 ksf)	75 – 150	12000 – 24000
$200 < q_u < 400$ k Pa	150 – 300	24000 – 48000
$q_u > 800$ k Pa	> 300	> 48000

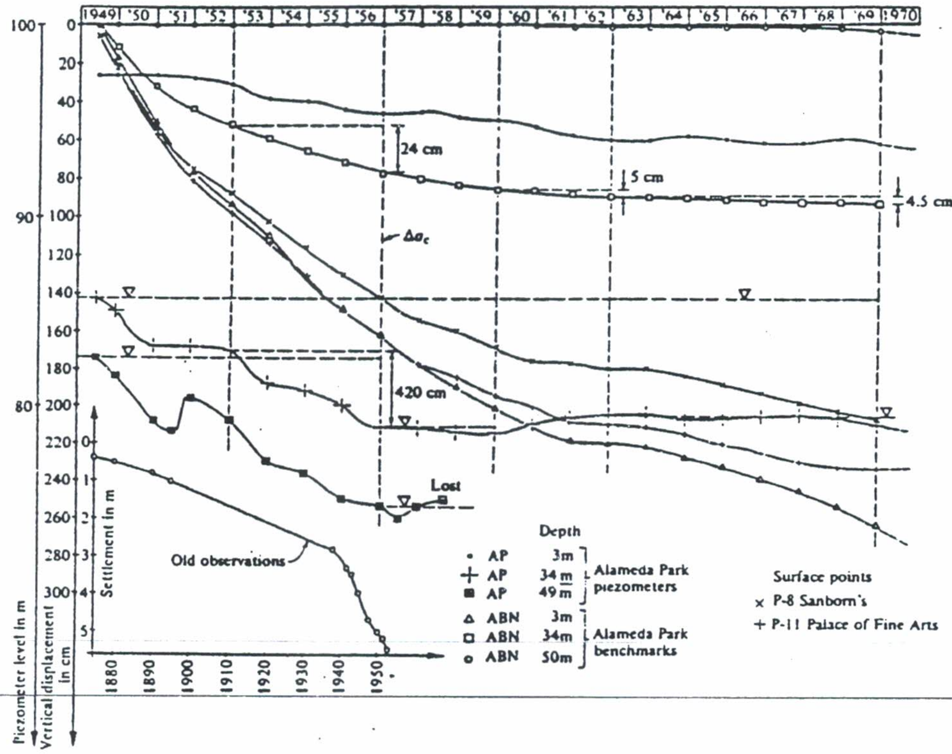


Fig. 12-V.3 Observed settlements with respect to ABN-49 benchmark in Alameda Park, Mexico City.

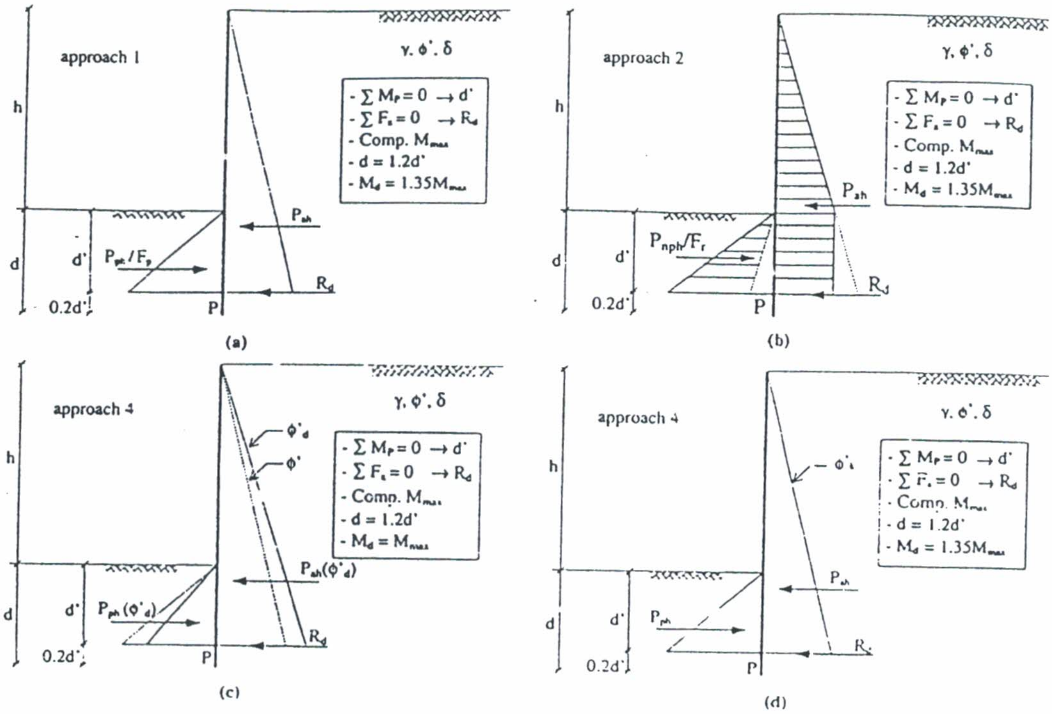


Figure 2. Approaches for the introduction of safety in the design of flexible cantilever retaining walls: (a) dividing the passive thrust by F_p; (b) dividing the net passive thrust by F_r; (c) EC 7 – Case C; (d) EC 7 – Case B.

Table 1. (ENV 1997-1:1994). Partial factors – ultimate limit states in persistent and transient situations.

	Actions (γ_F)			Ground Properties (γ_M)		
	Permanent (γ_G)		Variab. (γ_Q)			
Case	Unfav.	Fav.	Unfav.	tan ϕ'	c'	c_u
A	1.00	0.95	1.50	1.10	1.20	1.20
B	1.35	1.00	1.50	1.00	1.00	1.00
C	1.00	1.00	1.30	1.25	1.60	1.40

Note: Case A is only relevant to buoyancy problems, where hydrostatic forces comprise the main unfavourable action.

approach 3 – an alternative method consists of increasing the embedded wall length by a factor F_d; a value of 1.3 is normally adopted;

approach 4 – partial safety factors according EC 7 (see Table 1) can also be applied; if live loads at the surface are assumed as null, Case C corresponds to consider design values of permanent actions equal to the characteristic ones ($\gamma_G = 1$), together with design values of the soil parameters obtained on the basis of partial safety factors γ_M greater than 1.0 (Fig. 2c); for Case B, characteristic strength parameters are used in combination with a factor of 1.35 to multiply permanent actions or their effect (Fig. 2d).

In order to compare results of the maximum wall bending moment, the values obtained from the approaches 1 to 3, which can be considered characteristic values, are multiplied by 1.35.

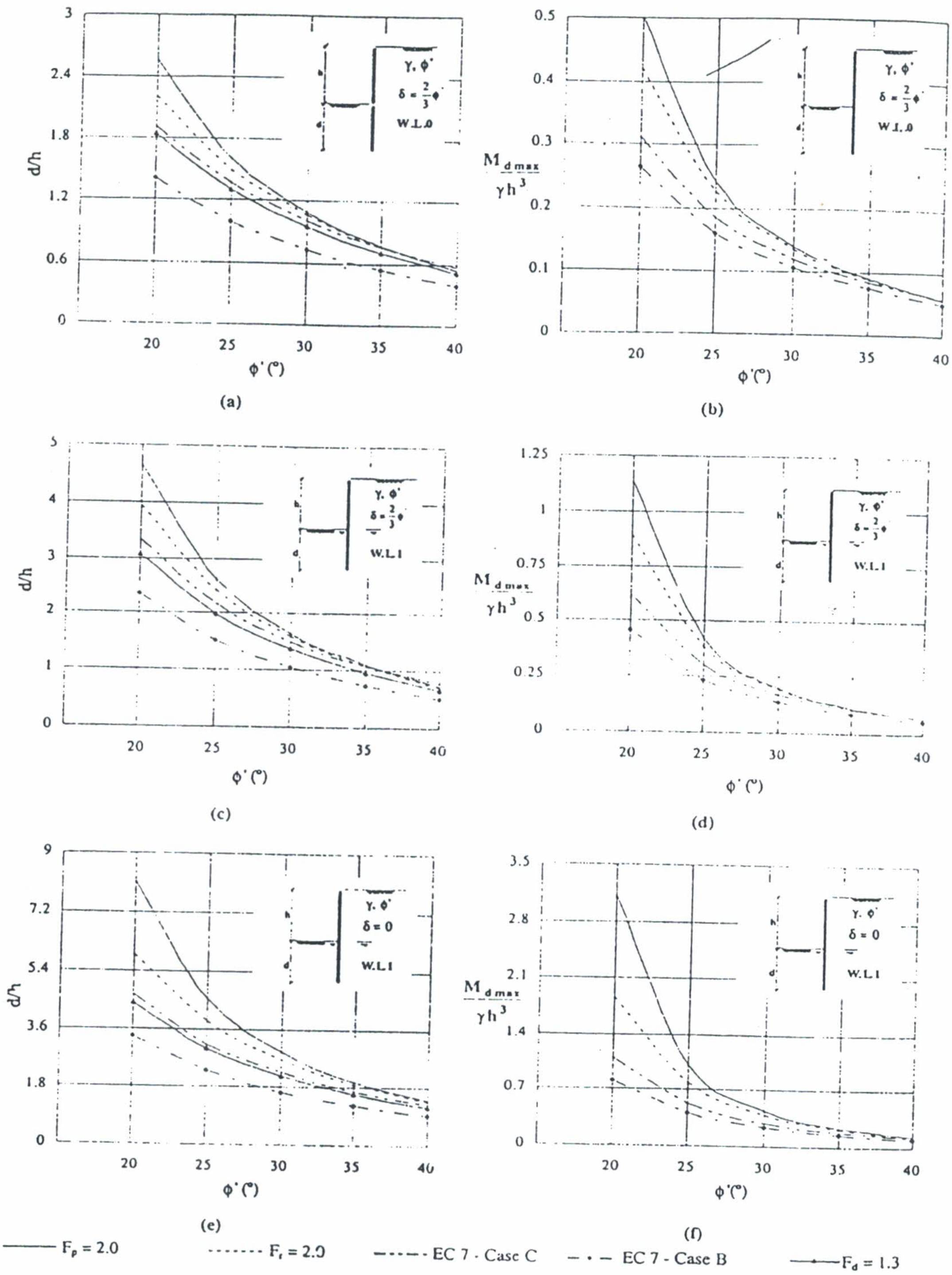


Figure 3. Embedded wall height and design wall bending moment: (a,b) water level below the wall tip and $\delta = (2/3)\phi'$; (c,d) water level at the base of the cut and $\delta = (2/3)\phi'$; (e,f) water level at the base of the cut and $\delta = 0$.

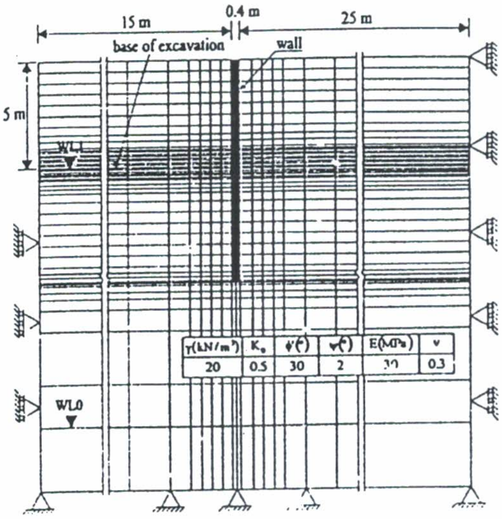


Figure 4. Finite element mesh and main assumptions of the numerical case study.

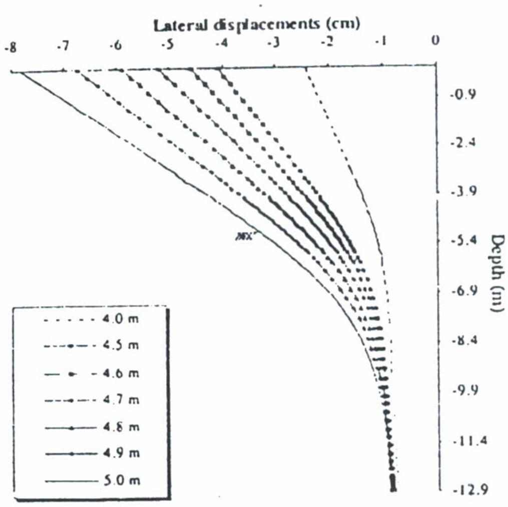


Figure 5. Wall designed according to *approach 2* (W.L.1, $\delta = (2/3)\phi'$): evolution of the lateral wall displacements.

$\delta = (2/3)\varphi'$. They show, respectively, the evolution of the lateral wall displacements, the distribution of the final wall bending moments and the effective normal earth pressures on both sides of the wall at the end of the excavation (5 m).

The comparison of the maximum bending moments with the values provided by the limit equilibrium method will be done later.

In Figure 7 the dashed lines correspond to the horizontal components of the theoretical active (behind the wall) and passive (in front of the wall) earth pressures,

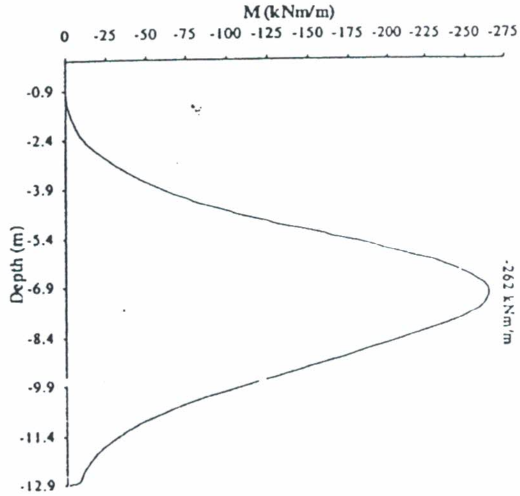


Figure 6. Wall designed according to *approach 2* (W.L.1, $\delta = (2/3)\phi'$): distribution of the final wall bending moments.

or to their algebraic addition, computed for the values of δ effectively mobilized in the analyses. These values, which are shown on the left graphs, are close to the one assumed in the limit equilibrium method (20°). Typically, the mobilized values of δ in front of the wall are greater than the ones at the opposite side.

3.3 *Wall designed according to Eurocode 7*

3.3.1 *General*

As was noted before, Case B provides an embedded wall height that corresponds to a safety factor marginally greater than 1.0 in relation to a limit state of rotation around the wall tip. Since the wall height will never be conditioned by this case, the finite element analyses presented in this paragraph considered the wall geometry provided by the limit equilibrium method explained in Figures 1 and 2 combined with the safety factors of Case C.

Two types of analyses have been carried out:

- *analyses CC*, i.e. analyses in which the wall height is calculated as referred above and characteristic permanent loads and design values of the strength parameters of the ground according to Case C are considered;
- *analyses CB*, i.e. analyses with wall height obtained in the same way as before, taking characteristic permanent loads and characteristic values of soil properties, and whose results in terms of internal structural forces are multiplied by 1.35 (this corresponds to Case B).

This two types of analyses seem to be the proper way of applying finite element models to this problems of

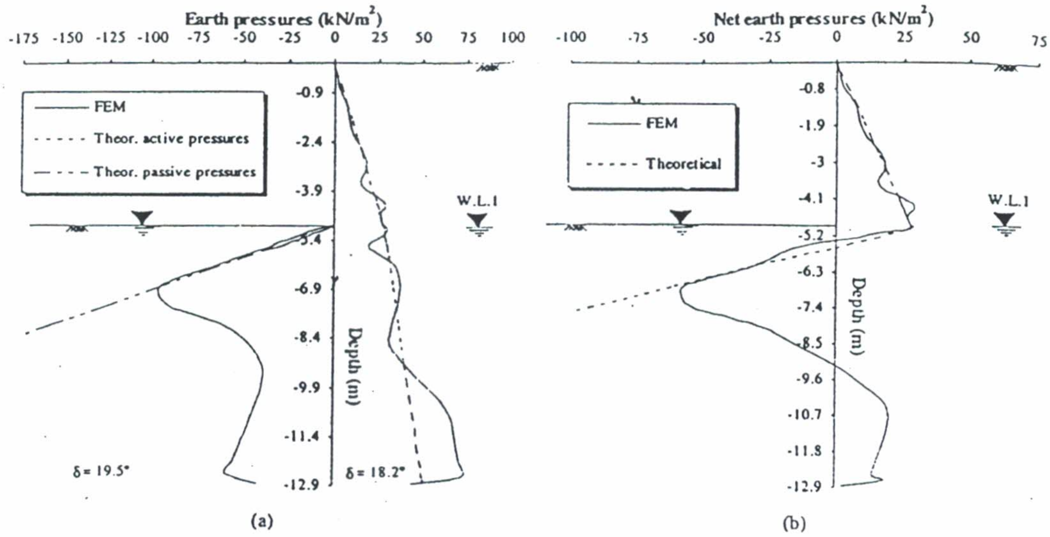


Figure 7. Wall designed according to *approach 2* (W.L.1, $\delta = (2/3)\phi'$): effective final normal earth pressures (a) on both sides of the wall and (b) showing the net pressures on the embedded height.

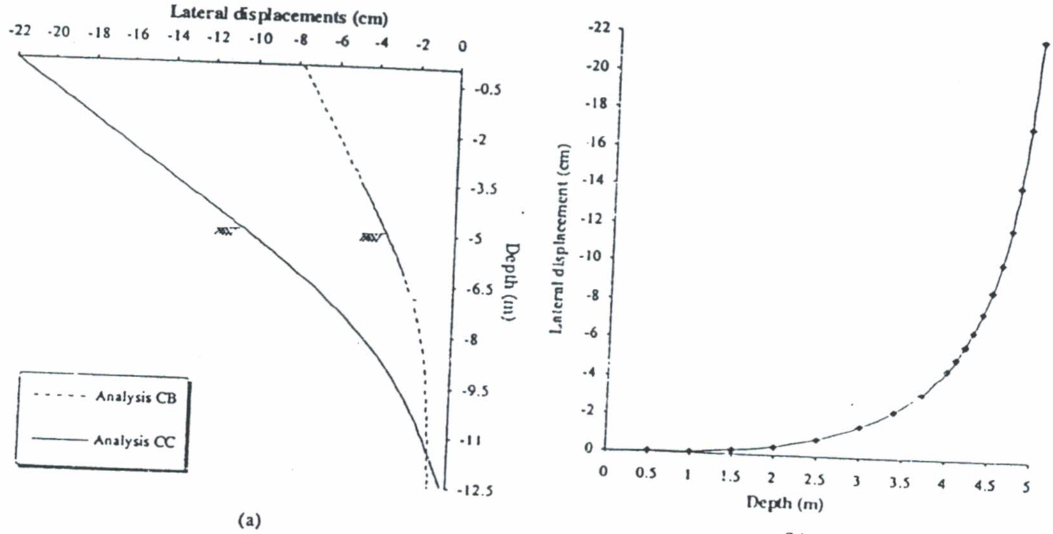


Figure 9. Lateral wall displacements (W.L.1, $\delta = (2/3)\phi'$): (a) final distribution from analyses CB and CC; (b) evolution of the displacement of the top of the wall with the current excavation depth in analysis CC.

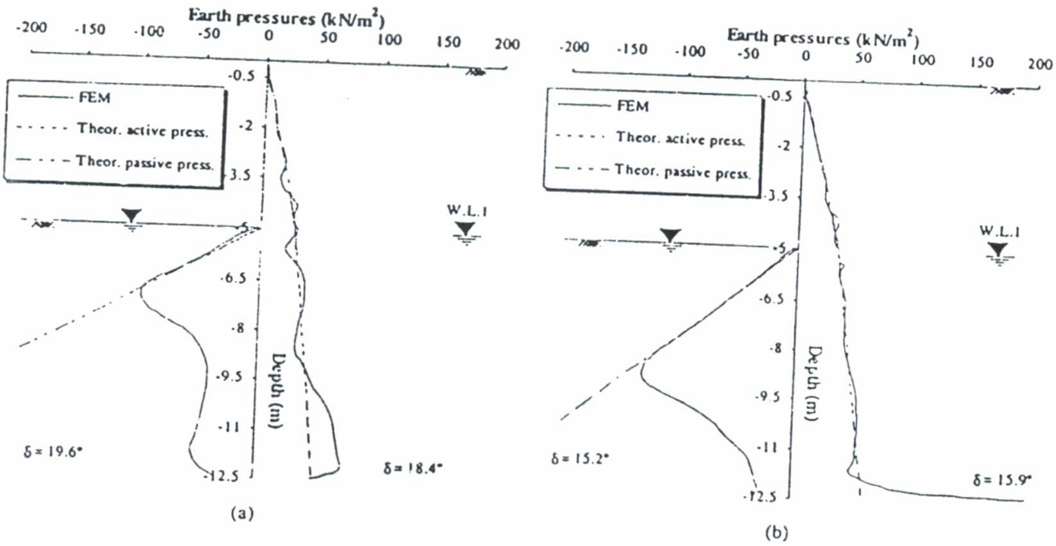


Figure 10. Effective final normal earth pressures on both sides of the wall (W.L.1, $\delta = (2/3)\phi'$): (a) *analysis* CB; (b) *analysis* CC.

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Referensi research :

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JURUSAN TEKNIK SIPIL
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INSTITUT SANIS DAN TEKNOLOGI NASIONAL

Nomor : 168 /03.1-Es/XI/2002
Lamp. :
Hal : **Pembicara Kuliah Umum.**

Kepada yang terhormat,
Bapak Ir. H. Idrus, MSc
Di
Jakarta

Dengan hormat,

Sehubungan dengan akan diselenggarakan kegiatan Kuliah Umum di Jurusan Teknik Sipil - Fakultas Teknik Sipil dan Perencanaan – Institut Sains dan Teknologi Nasional yang akan kami selenggarakan pada :

Hari / Tanggal	: Kamis, 28 November 2002
Jam	: 10.00 s/d selesai
Ruang	: B.1 – FTSP - ISTN

Kami mengharapkan kesediaan Bapak untuk mengisi acara tersebut sebagai pembicara.

Demikian pemberitahuan kami, atas perhatian dan kerjasamanya kami ucapkan terima kasih.

Jakarta, 22 November 2002
Ketua Jurusan Teknik Sipil,

JURUSAN TEKNIK SIPIL

Ir. Harry Hartawan, MCM