SOIL MECHANICS II
CIVL 311
COURSE NOTES
2010

Module 7
Design of Shallow Foundations

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Module 7

Design of Shallow Foundations

Overall Learning Objectives

- Ultimate Limit States – estimation of limit load by theoretical and empirical means (Book 2 pp. 200 – 224, 234-235)
- Serviceability Limit State – estimation of settlements of foundations on clay (Book 2 pp.224-234)
- Serviceability Limit State – estimation of settlements of foundations on sand (Book 2 pp. 235-240)
- Design Issues and Procedures (Book 2 pp. 251 – 270)

SOIL MECHANICS II
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Module 7.1
Ultimate Limit States
Estimation of limit bearing capacity
by theoretical and empirical means

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Working Stress Design

This is the conventional approach in foundation engineering:

i. Bearing pressure < Ultimate bearing capacity

ii. Estimate Settlements: If calculated total and differential settlements exceed acceptable limits, reduce pressure until settlements are acceptable.

iii. The final design pressure is called the allowable bearing pressure.

Selection of allowable bearing pressure

Four basic approaches:

Presumed values –based on experience of allowable bearing pressures which are known to have satisfied both strength and deformation criteria

Direct approach - directly from an index test such as the SPT to allowable bearing pressure. Gives a bearing pressure that will limit settlements to less than 25 mm.

Indirect approach - based on theoretical equations, ultimate bearing pressure calculated using Bearing Capacity theory (we covered this under Module 1.4). Effective stress/total stress approaches.

Load Testing – a full-scale footing is tested, preferably to failure.

Presumed Values

TABLE 9.3 *Presumed Preliminary Design Bearing Pressure (continued)*

Group	Types And Conditions Of Rocks And Soils	Strength Of Rock Material	Preliminary Design Bearing Pressure (kPa)	Remarks
Fine-grained soil	Very stiff to hard clays or heterogeneous mixtures such as till		300-600	Fine-grained soils are susceptible to long-term consolidation settlement due to imposed loads and are often susceptible to severe swelling or shrinking due to changed moisture conditions. If the Plasticity Index (I_p) exceeds 30 and the clay content exceeds 25 %, the long-term performance of the foundation may be significantly affected by swelling or shrinking of the subsoils, and a complete assessment of these possibilities is necessary as discussed in Chapter 15
	Stiff clays		150-300	
	Firm clays		75-150	
	Soft clays and silts		<75	
	Very soft clays and silts		not applicable	
Organic Soils	Peat and organic soils		Not applicable	
Fill	Fill		Not applicable	

Notes:
1. The above values for sedimentary or foliated rocks apply where the strata or the foliation are level or nearly so, and, then, only if the area has ample lateral support. Tilted strata and their relation to nearby slopes or excavations should be assessed by a person knowledgeable in this field of work.
2. Sound rock conditions allow minor cracks at spacing not closer than 1 m.
3. To be assessed by examination in-situ, including test loading if necessary.
4. These rocks are apt to swell on release of stress, and on exposure to water they are apt to soften and swell.
5. The above values are preliminary estimates only and may need to be adjusted upwards or downwards in a specific case. No consideration has been made for the depth of embedment of the foundation. Reference should be made to other parts of the Manual when using this table.

From Canadian Foundation Engineering Manual, 4th Edition. Canadian Geotechnical Society

Also see Section 7.6 – Budhu Text

Caution:
The above pressures are useful for preliminary design, but should be confirmed by a site-specific geotechnical evaluation.

From in situ tests

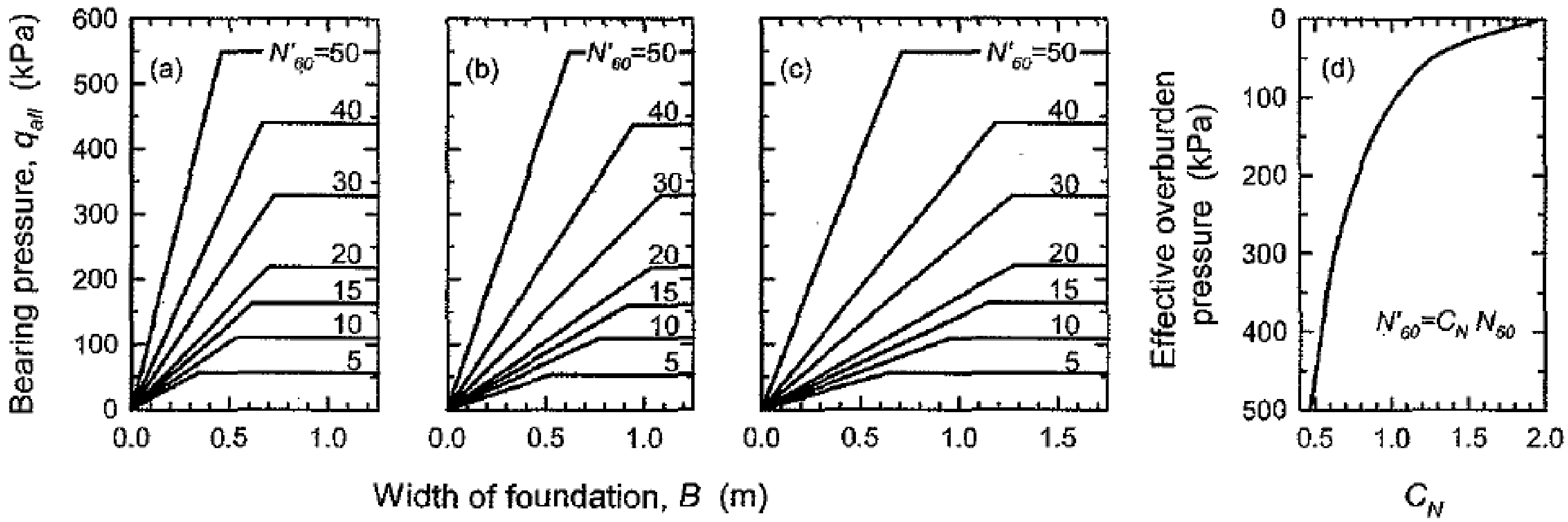

FIGURE 11.8 *Design bearing pressure for foundations on sand for settlement not exceeding 25 mm based on SPT-N results for: (a) D/B=1, (b) D/B=0.5, and (c) D/B=0.25. SPT-N value from field to be modified by factor C_N given in (d) for use in (a)-(c). Modified from Peck et al. (1974)*

The design bearing pressures obtained from Figure 11.8 above are applicable for low-risk projects and have been shown to be generally conservative, resulting in foundations which experience settlements of less than 25 mm.

The above design methods apply to vertically loaded foundations on flat ground.

Also, refer to Section 7.8 of Budhu text

Indirect Approach
(already completed under Module 1.4)

Effective Stress Analysis (ESA)

$$q_u = \gamma D_f (N_q - 1)(s_q d_q i_q b_q g_q r_q w_q) + 0.5\gamma B' N_\gamma (s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma r_\gamma w_\gamma)$$

Total Stress Analysis (TSA)

$$q_u = 5.14 s_u (s_c d_c i_c b_c g_c r_c)$$

Note : B' = Effective Footing Width (discussed later)

Definition of the factors to account for the different variations :

s_q, s_γ, s_c - Shape of Footing

d_q, d_γ, d_c - Depth of Footing Embedment

i_q, i_γ, i_c - Inclination of Load on Footing

b_q, b_γ, b_c - Inclination of the Base of Footing

g_q, g_γ, g_c - Inclination of the Ground Surface at the Footing Location

r_q, r_γ, r_c - Soil Compressibility

w_q, w_γ - Groundwater level

Load Testing

www.ecs.umass.edu/geotech/nges.html

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Module 7.2
Serviceability Limit State
Estimation of settlements

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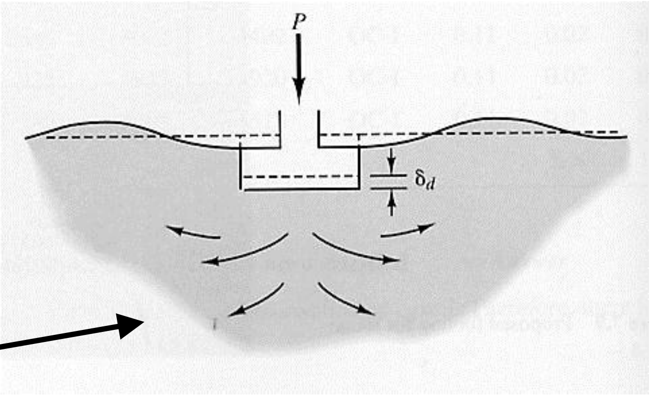
General Design Principles

- For well-designed foundations, strains in the ground are typically in the range of 0.001% to 1%.

- In ground response modeling, the stresses and strains are calculated using the theory of elasticity.

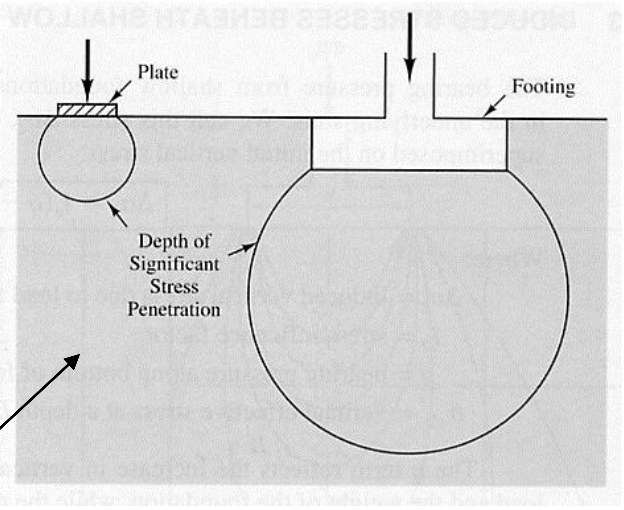
- When such calculated movements are compared to full-scale load test results:
    - adjustments are required to account for the difference between real behaviour and the simplifying elasticity theory assumptions.
    - various methods have evolved for coarse-grained and fine-grained soils – Budhu 7.7 and notes below

Nature of settlements



- Immediate (distortion)
  - due to distortion under imposed shear stresses (occurs without change in void ratio or drainage of water from pores, i.e. undrained in fine grained soils)
- Time dependent
  - due to compression of the soil skeleton (consider magnitude and rate of settlement)
  - secondary compression

Settlement Estimation Methods



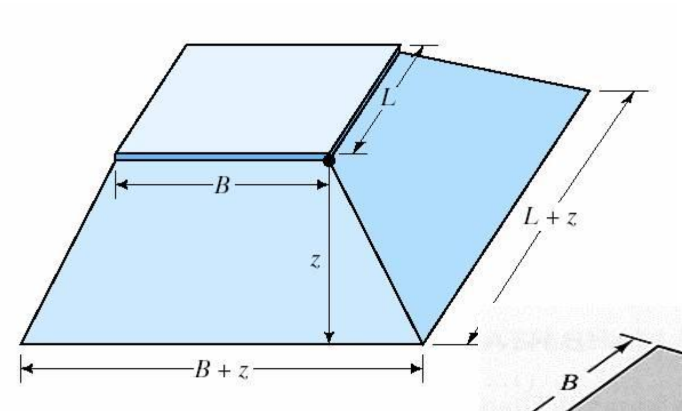
- Plate load tests (tests only a small volume of soil)
- Analysis based on laboratory tests and consolidation theory
- Analysis based on in-situ tests

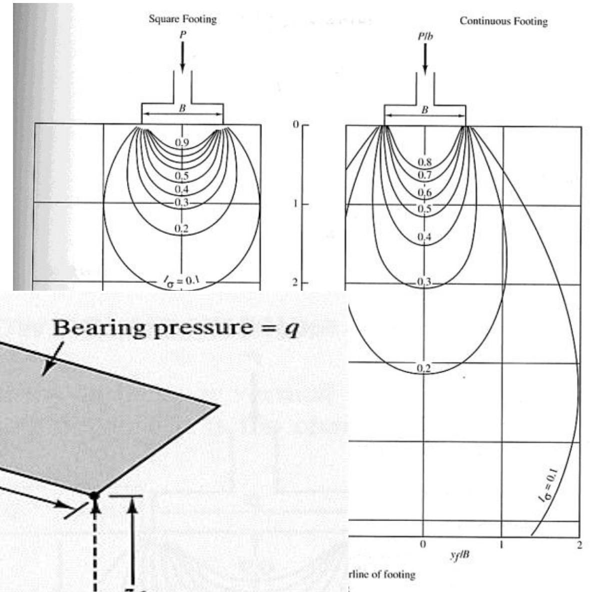
Analytical approach

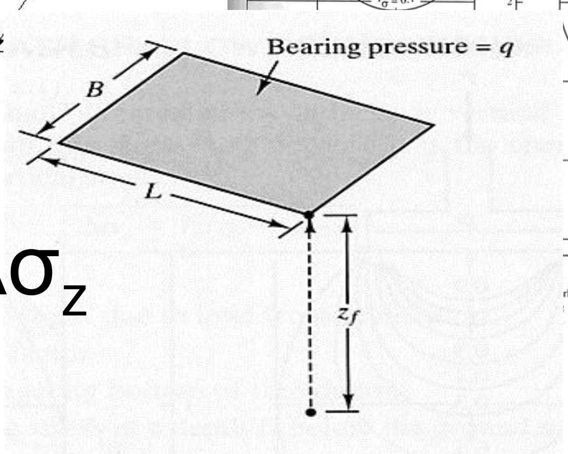
- Estimate induced stresses
  - Boussinesq – Newmark method
  - Simplified method
- Estimate settlements using theory

Estimation of Stresses

$\Delta\sigma_z$ = Influence factor * Net Increase in Load







$\sigma'_{fin} = \sigma'_{zo} + \Delta\sigma_z$

See previous modules on stresses in soils

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Module 7.2.1
Serviceability Limit State
Estimation of settlements of foundations on clay

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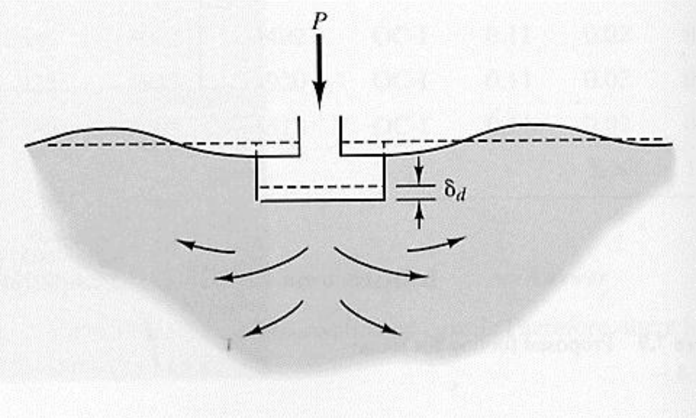
Estimation of Settlement in saturated fine-grained soils (e.g., clay)

1. Total settlement made up of three components
   1. settlements due to distortion
   2. settlements due to consolidation (i.e., dissipation of excess pore water pressures)
   3. settlements due to secondary compression

Estimation of Distortion Settlements in clay

$$\boxed{\rho_e = \frac{(q - \sigma'_{zD})B}{E_u} I_1 I_2}$$

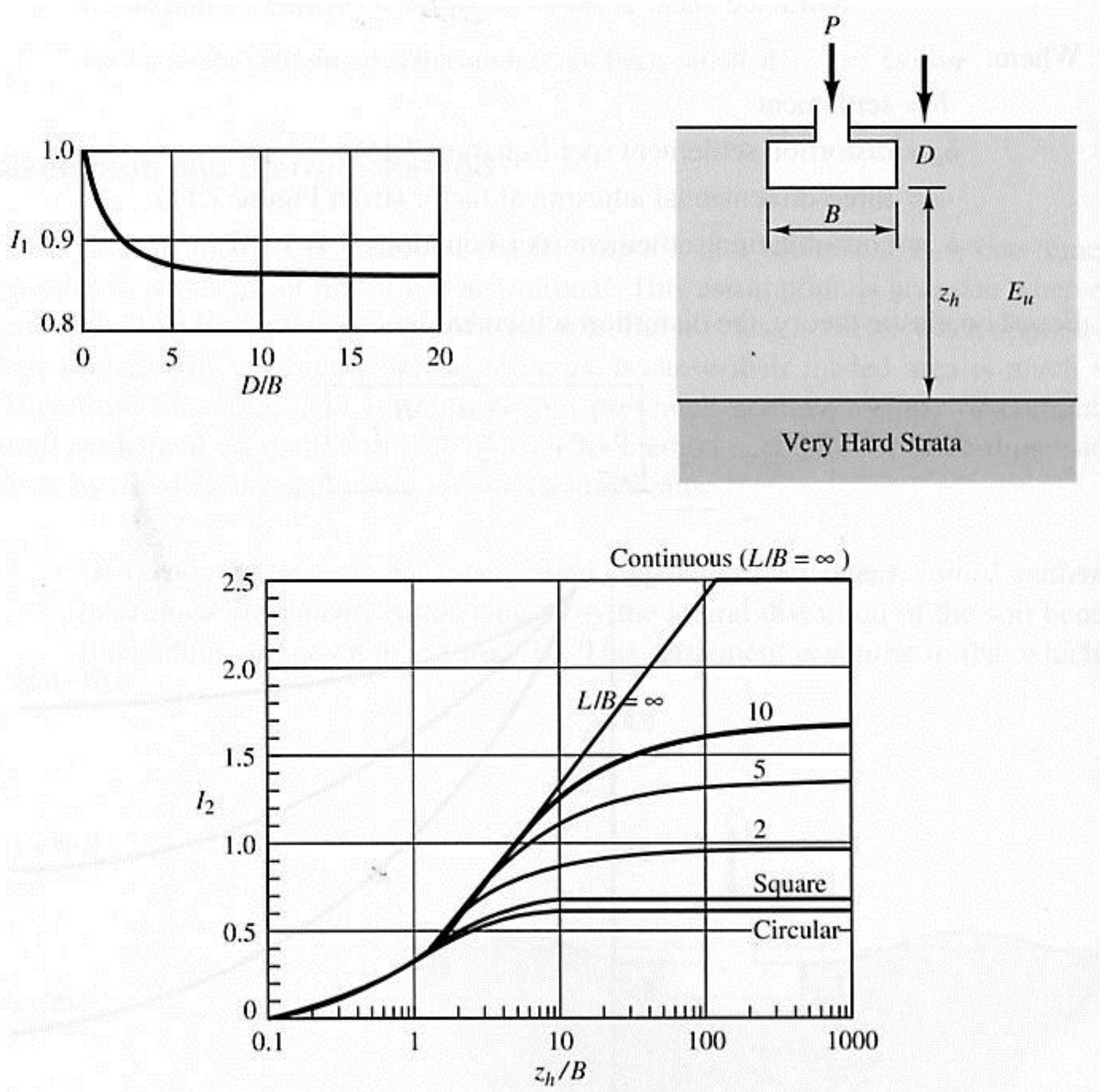


Where:

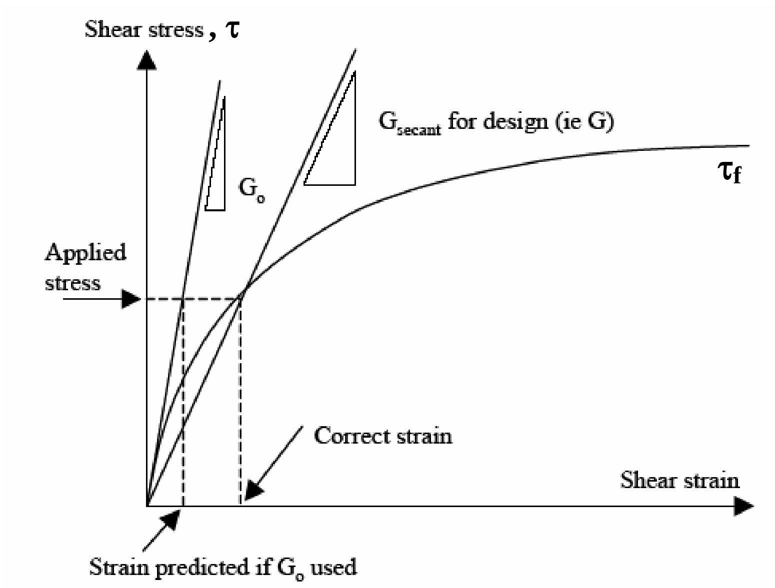
ρ_e = distortion settlement
q = bearing pressure
σ_{zD}' = vertical effective stress at a depth D below the ground surface
B = foundation width
I_1, I_2 = influence factors (per Figure 7.12)
E_u = undrained modulus of elasticity of soil

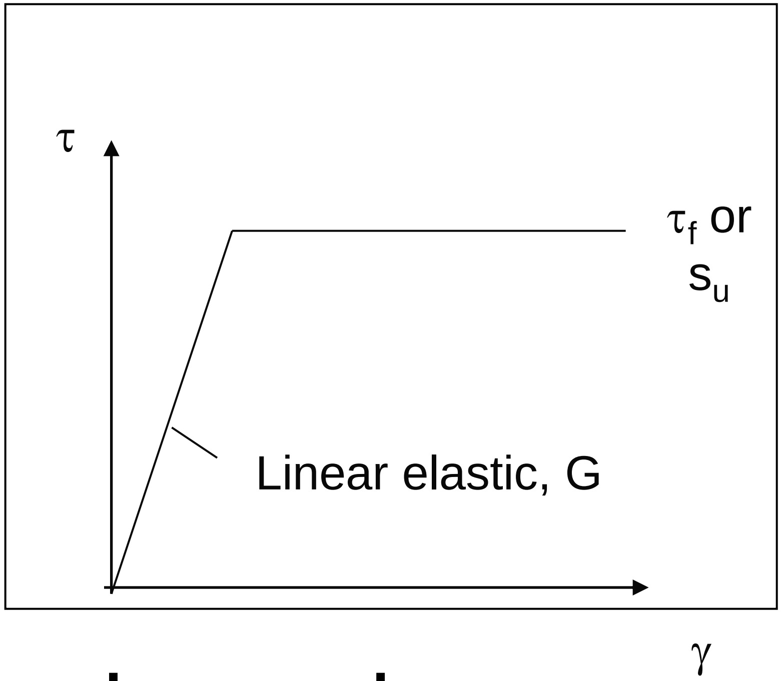
Implicitly assumes ν=0.5

I_1 and I_2 for Distortion Settlement calculation



Idealization of soil behaviour (estimation of E_u)





Strength and stiffness depend on current stress, density or void ratio and on stress history

Estimation of Consolidation Settlements (ρ_{pc}) in fine-grained soils

- In most instances, need to subdivide the clay layer in zone of stress influence;

- Estimate initial vertical effective stress profile. This information will provide σ'_{zo} for mid-point of each layer;

- Using $\Delta\sigma_z$ calculated at different depths, estimate final stress profile. (i.e., determine $\sigma'_{fin}=\sigma'_{zo}+\Delta\sigma$ for mid-point of each layer);

- Estimate yield stress (σ'_c) profile in zone of influence;

- Knowing $\sigma'_{fin} < \sigma'_c$, or $\sigma'_{fin} > \sigma'_c$, for mid-point of each layer, compute consolidation settlements (ρ_{pc}) for each layer using 1-D consolidation theory (equations from Module 1.1 is given on the next page as a reminder)

$$\boxed{\sigma'_{\text{fin}} = \sigma'_{zo} + \Delta\sigma_{\text{z}}}$$

6.4.2 Primary Consolidation Settlement of NC Fine Grain (p. 177 Budhu, Wiley)

$$\rho_{pc} = H_o \frac{\Delta e}{1+e_o} = \frac{H_o}{1+e_o} C_c \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zo}}}; \quad \text{OCR} = 1 \qquad (6.14)$$

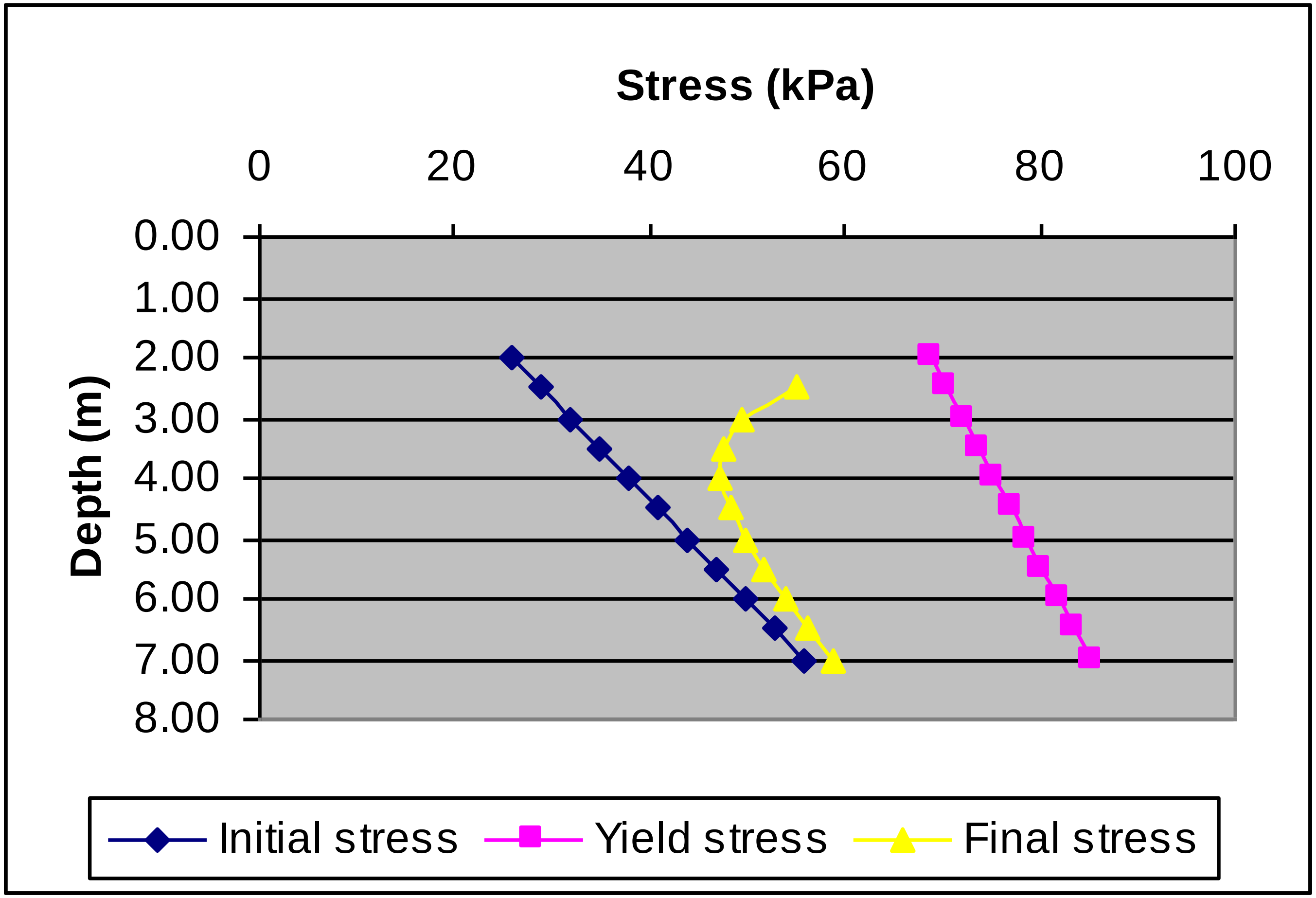
6.4.3 Primary Consolidation Settlement of OC Fine Grain (p. 177 Budhu, Wiley)

$$\rho_{pc} = \frac{H_o}{1+e_o} C_r \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zo}}}; \quad \sigma'_{\text{fin}} < \sigma'_{\text{zc}} \qquad (6.15)$$

$$\rho_{pc} = \frac{H_o}{1+e_o} \left(C_r \log \frac{\sigma'_{\text{zc}}}{\sigma'_{\text{zo}}} + C_c \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zc}}} \right); \quad \sigma'_{\text{fin}} > \sigma'_{\text{zc}} \qquad (6.16)$$

$$\rho_{pc} = \frac{H_o}{1+e_o} \left[C_r \log\left(\text{OCR}\right) + C_c \log \frac{\sigma'_{\text{fin}}}{\sigma'_{\text{zc}}} \right]; \quad \sigma'_{\text{fin}} > \sigma'_{\text{zc}} \qquad (6.17)$$

Example



$\sigma'_{zf} < \sigma'_c$ throughout zone of influence

$\therefore$ use elastic approach to estimating settlement

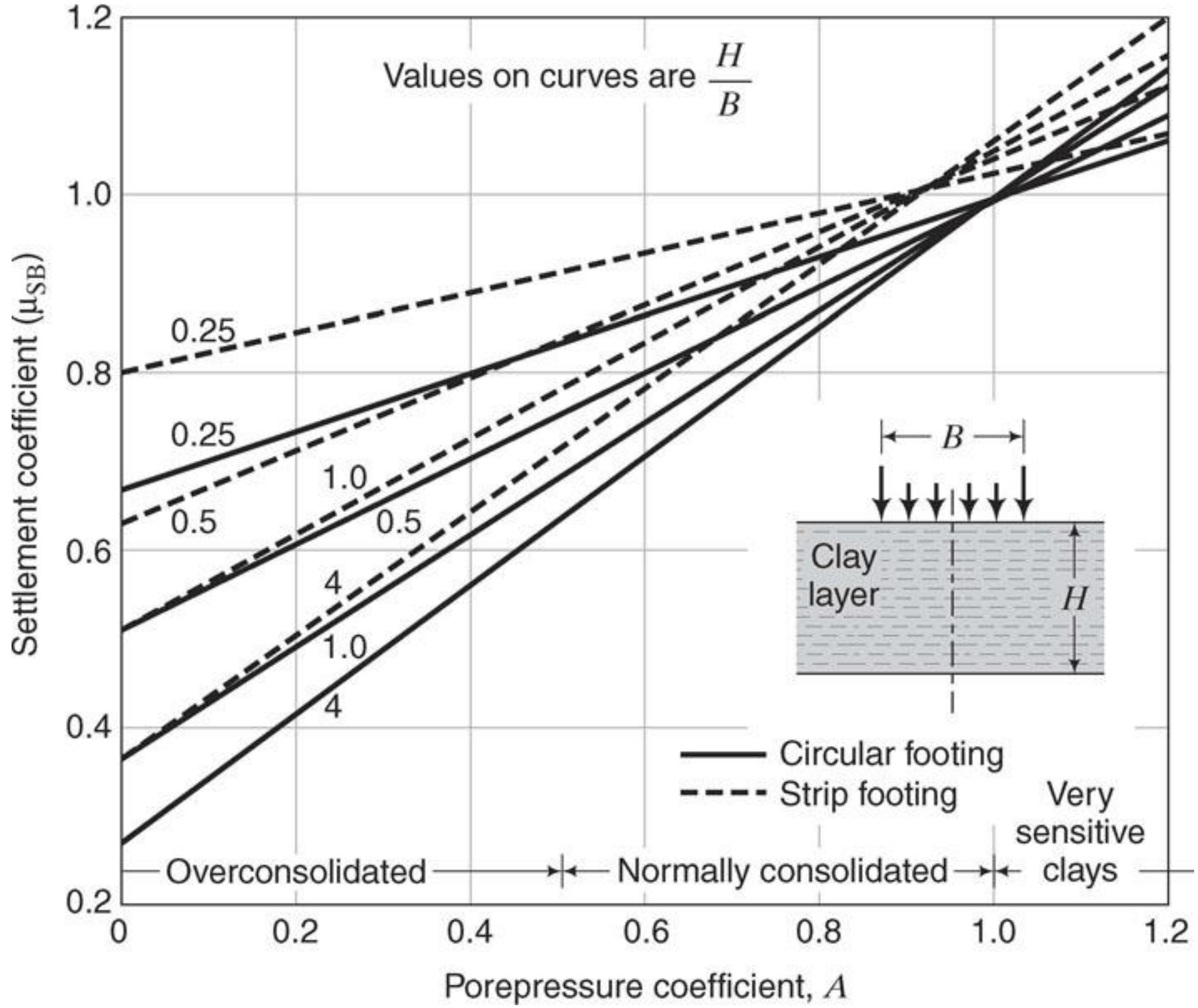
Correction of calculated 1D consolidation Settlementssin fine-grained soils for

3D effects

- Estimate consolidation settlement, ρ_{pc}, using theory of 1D consolidation
- Correct calculated consolidation settlement for 3D effects using Skempton and Bjerrum correction, μ_{SB}

$$\rho_{pc,3D} = \mu_{SB}\rho_{pc,1D}$$

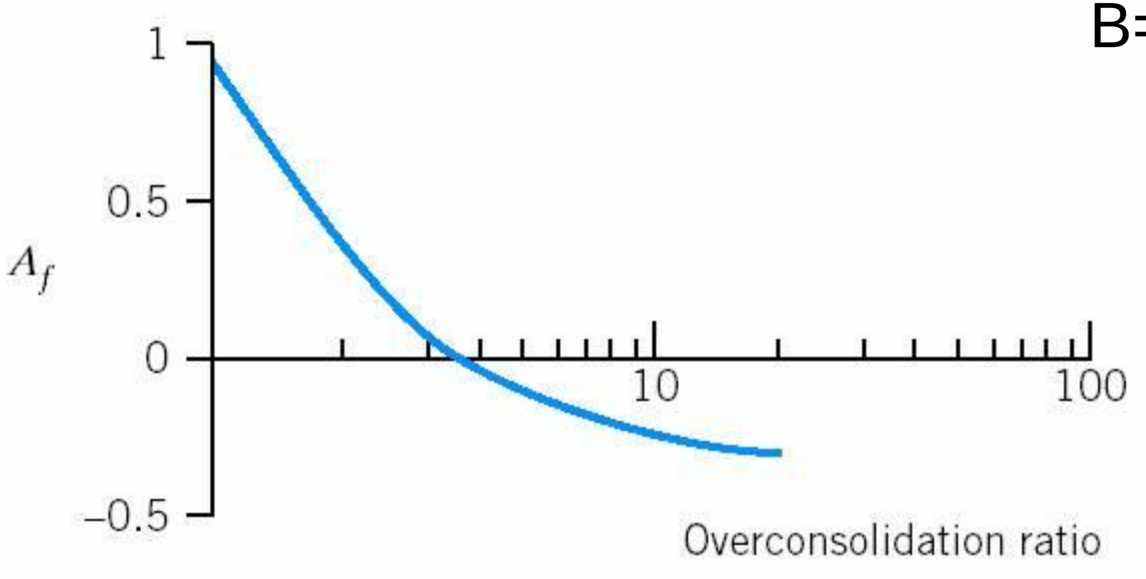
Skempton and Bjerrum



A_f varies with OCR

$$\Delta u = B\left[\Delta\sigma_3 + A\left(\Delta\sigma_1 - \Delta\sigma_3\right)\right]$$

B=1 for saturated soil



Estimation of secondary compression settlements in clay

- compute secondary compression settlements for each layer (equations from Module 1.1 is given below as a reminder)

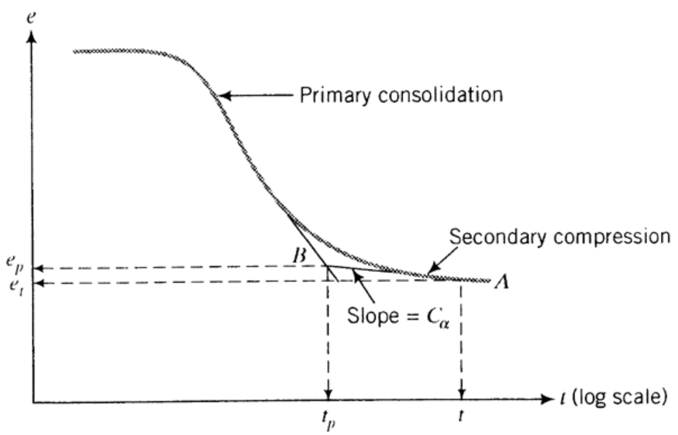


FIGURE 6.11 Secondary compression.

$$C_a = -\frac{(e_i - e_p)}{\log(t/t_p)}; t > t_p$$ Equation 6.41

$$\rho_{sc} = \frac{H_0}{(1+e_p)} C_\alpha \log(t/t_p)$$ Equation 6.42

secondary compression settlements are generally not important for OC soils.

Total settlements - Clay

- Total settlement, ρ, in fine-grained soils (clay), is given by:

$$\rho = \Sigma(\rho_e + \mu_{SB}\rho_{pc,1D} + \rho_{sc})$$

  - In stiff, OC soils, where $\sigma'_{fin} < \sigma'_c$, settlements due to consolidation are small and distortional settlement is a large proportion of the final settlement
  - In NC soils, where $\sigma'_{z,f} > \sigma'_c$, consolidation settlements are significant

Conclusion on settlement calculations in saturated fine- grained soils

- Total settlement made up of three components
  - Distortional (constant volume) or Immediate
    - calculate by elasticity methods
  - Consolidation – or time dependent
    - Use 1-D consolidation theory but adjust for 3D effects using Skempton-Bjerrum factor
  - Secondary (long term)

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Module 7.2.2
Serviceability Limit State
Estimation of settlements of foundations on sand

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Estimation of settlements in coarse-grained soils (sands)

- Distortional and consolidation settlements happen concurrently (i.e., drained behaviour)
- 1D consolidation approach not applicable as:
  - cannot distinguish between undrained distortional settlements and settlements due to volume change in the soil.
  - In any case cannot get relatively undisturbed samples
- As such, settlements can be treated as distortion-induced
- Methods have evolved based on in situ tests

Settlement calculations based on elasticity theory

- General equation for settlement calculation strain influence factors is

$$\rho = \frac{qBI}{E_s}$$

  - Where q is bearing pressure
  - B is footing width
  - E_s is equivalent elastic modulus
  - I is Strain Influence Factor.

Derivation of influence factors

Based on
Mayne, P.W. and Poulos, H.G. “Approximate Displacement Influence Factors for Elastic Shallow Foundations”, Journal of Geotechnical and Geoenvironmental Engineering Vol 125, No.6, June 1999.

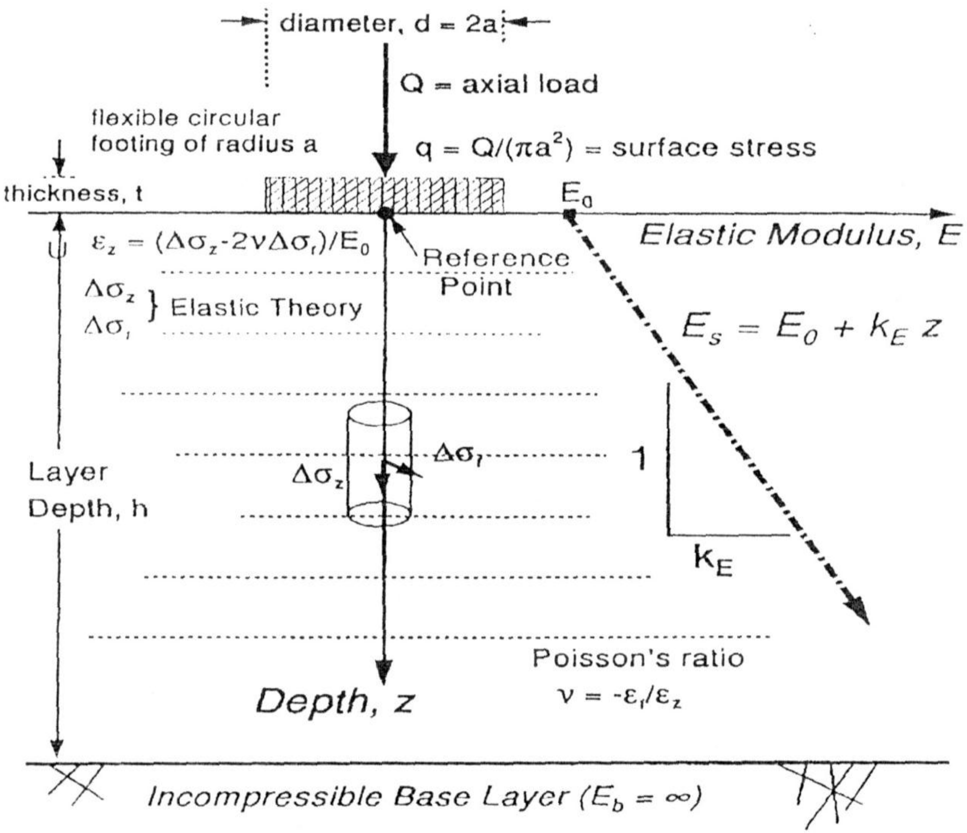


FIG. 1. Nomenclature Used in Development of Displacement Influence Factors

$I = \int \varepsilon_z dz^*$ where

ε_z is vertical strain

Z^* is z/B

B is footing width

Typical strain influence factors

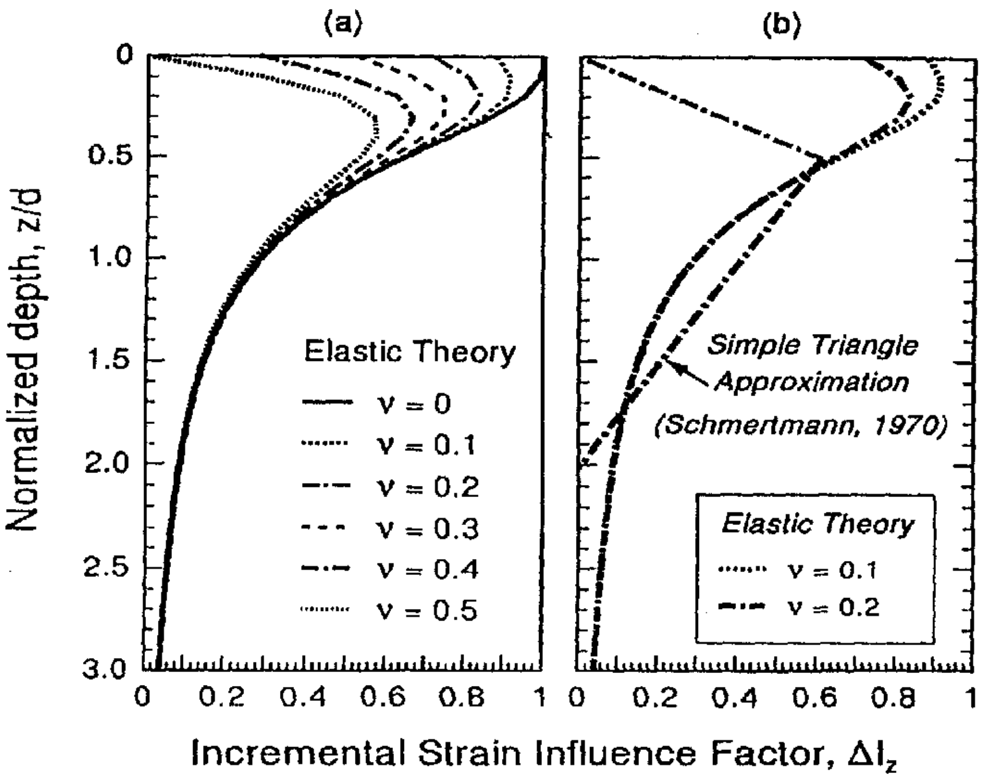
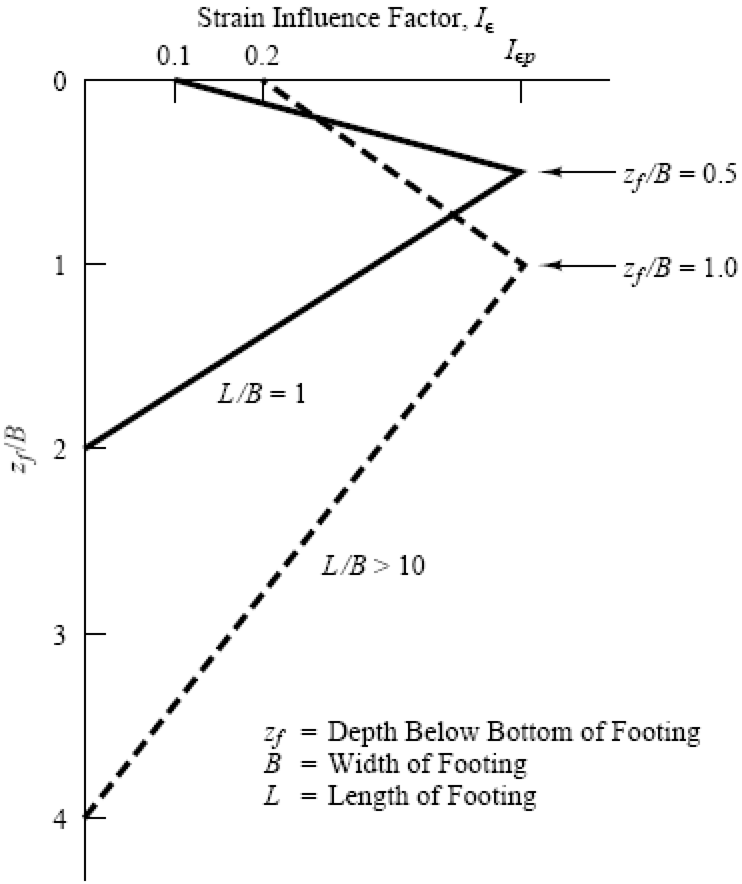


FIG. 2. Strain Influence Factors from: (a) Elastic Theory: (b) Simple Triangular Case

Schmertmann's 1978 method



Peak value of I_ε:

$I_{\varepsilon p}=0.5+0.1\sqrt{[(q-\sigma'_{zd})/\sigma'_{zp}]}$

σ'_{zp}= value of vertical effective stress at I_{ep}

σ'_{zd}=vertical effective stress at foundation level

q=bearing pressure

Schmertmann's Method

Used to compute settlement of spread footings on sandy soil

(1) Get modulus E_s from in-situ test
- CPT, SPT, Dilatometer, pressuremeter, etc.

(2) Calculate
- Strain influence factor,
- Depth factor - C_1
- Creep factor – C_2
- Shape factor – C_3

(3) Combine to calculate settlement

Schmertmann's Method

$$\rho = C_1 C_2 C_3 (q - \sigma'_{zD}) \Sigma \frac{I_\varepsilon H}{E_s}$$

$$\boxed{C_1 = 1 - 0.5\left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)}$$

$$\boxed{C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right)}$$

$$\boxed{C_3 = 1.03 - 0.03\, L/B \geq 0.73}$$

Where:

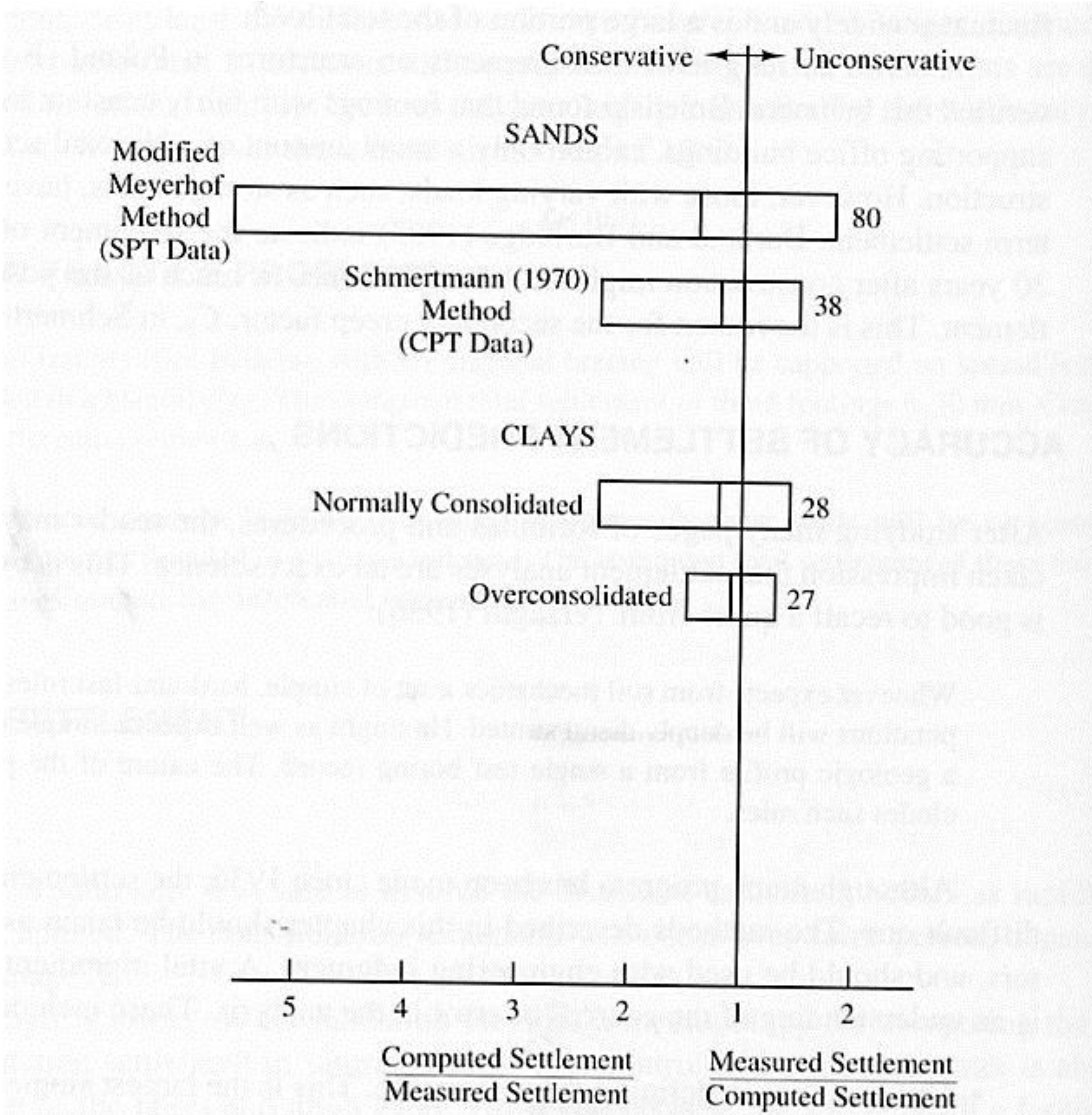
δ = settlement of footing
C_1 = depth factor
C_2 = secondary creep factor (see discussion in Section 7.8)
C_3 = shape factor = 1 for square and circular foundations
q = bearing pressure
σ_{zD}' = effective vertical stress at a depth D below the ground surface
I_ϵ = influence factor at midpoint of soil layer
H = thickness of soil layer
E_s = equivalent modulus of elasticity in soil layer
t = time since application of load (yr) ($t \geq 0.1$ yr)
B = foundation width
L = foundation length

E_s from CPT

TABLE 7.3 ***E_S*-VALUES FROM CPT RESULTS** [Adapted from Schmertmann, et al. (1978), Robertson and Campanella (1989), and other sources.]

Soil Type	USCS Group Symbol	E_s/q_c
Young, normally consolidated clean silica sands (age < 100 years)	SW or SP	2.5–3.5
Aged, normally consolidated clean silica sands (age > 3000 years)	SW or SP	3.5–6.0
Overconsolidated clean silica sands	SW or SP	6.0–10.0
Normally consolidated silty or clayey sands	SM or SC	1.5
Overconsolidated silty or clayey sands	SM or SC	3

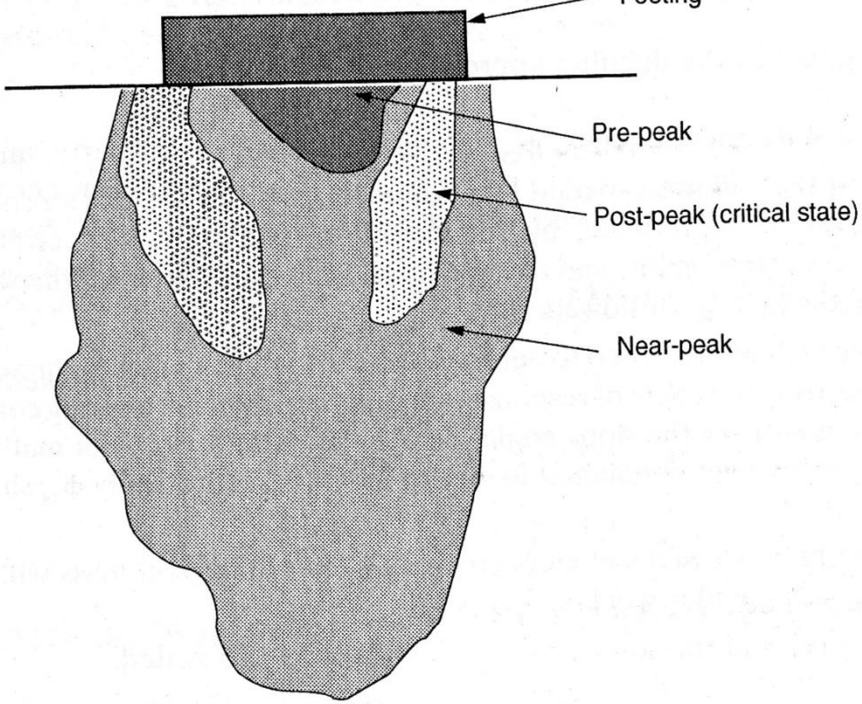
Accuracy of Settlement Predictions



Conclusion on settlements in coarse-grained soils (sands)

- In sands, use Schmertmann's method based on elasticity theory with stiffness estimated from in situ test data.
- Schmertmann's method relates E_s to Cone Penetration Resistance, q_c, in sands – easy to compute settlement
- Strain Influence approach appears to give reasonable results but very dependent on good choice of E_s.

Design for Ductility



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Module 7.3
Design Issues and Procedures

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Foundation Design Considerations

- Satisfy ultimate limit state
(bearing capacity)

- Satisfy servicebility limit state
(settlements)

- Constructability

- Economy

Ground Improvement

TABLE 7.7 Some Popular Ground Modification Methods

Method	Process	Soil type	Desired outcome
Vibro-compaction	Densification of soil by deep vibratory method	Coarse-grained soils mine-spoils	Reduce settlement, reduce potential for liquefaction, increase bearing capacity
Vibro-replacement stone—columns	Soil reinforced by stone columns	Coarse-grained soils, mine-spoils	
Vibro-displacement	Densification and reinforcement by installation of stone columns	All soils—best for coarse-grained soils	
Dynamic deep compaction	Heavy weight dropped on ground surface to densify soil	Coarse-grained soils	
Soil and rock anchors	Installation of anchors into soils	All soils	Lateral support of retaining walls, slope stabilization
Soil nailing	Installation of nails (steel bars) into soils	All soils except gravel and very soft clays	
Mini piles	Small diameter (5 to 50 mm diameter) piles to transfer structural loads to lower, stronger soil layer	All soil types	Reduce settlement, increase bearing capacity
Slurry walls	Slurry consisting of bentonite and water poured into excavations	All soils	Wall construction, drainage control, control of pollution migration, foundation construction
Grouting	Pumping of cementation material into soil and rocks (cement, bentonite, fly ash, lime, etc.)	All soils	Increase soil strength, reduce permeability, prevent collapse of coarse-grained soils and seepage groundwater control
Injection stabilization	Injection of a solution of water, lime slurry or potassium chloride into expansive soils	Fine-grained soil	Treatment of expansive soils

Ground Improvement and Drainage

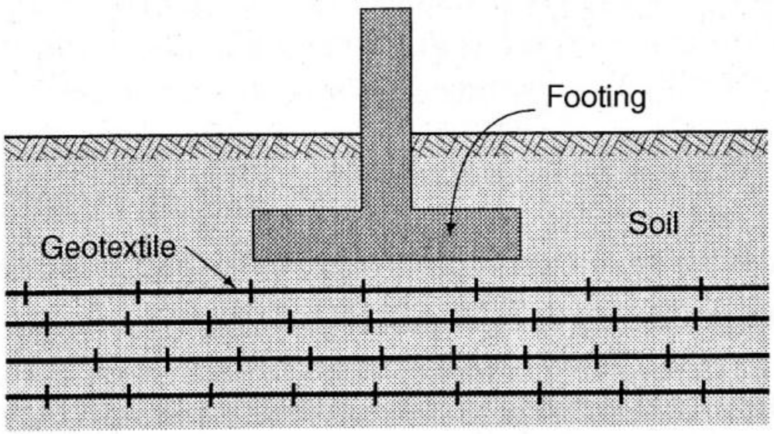


FIGURE 7.28 Geotextiles used to improve the bearing capacity of foundations.

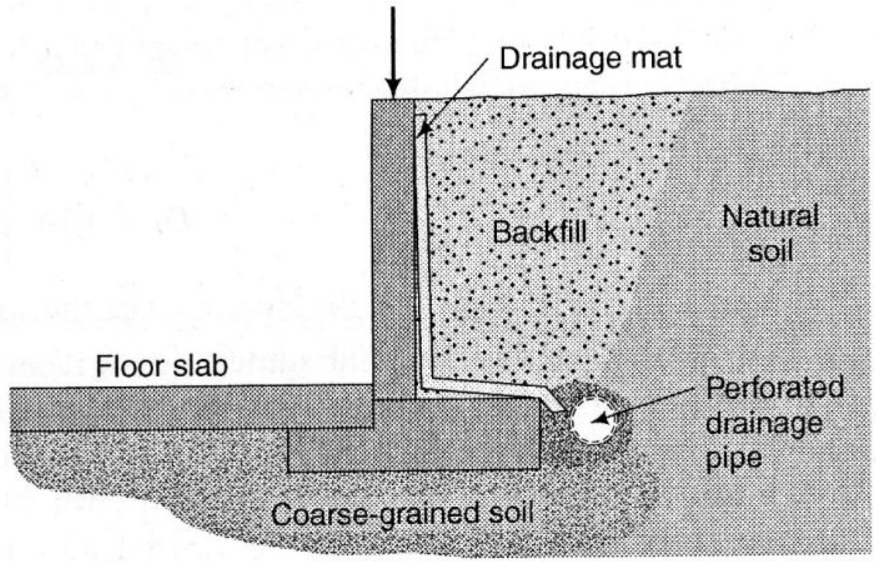
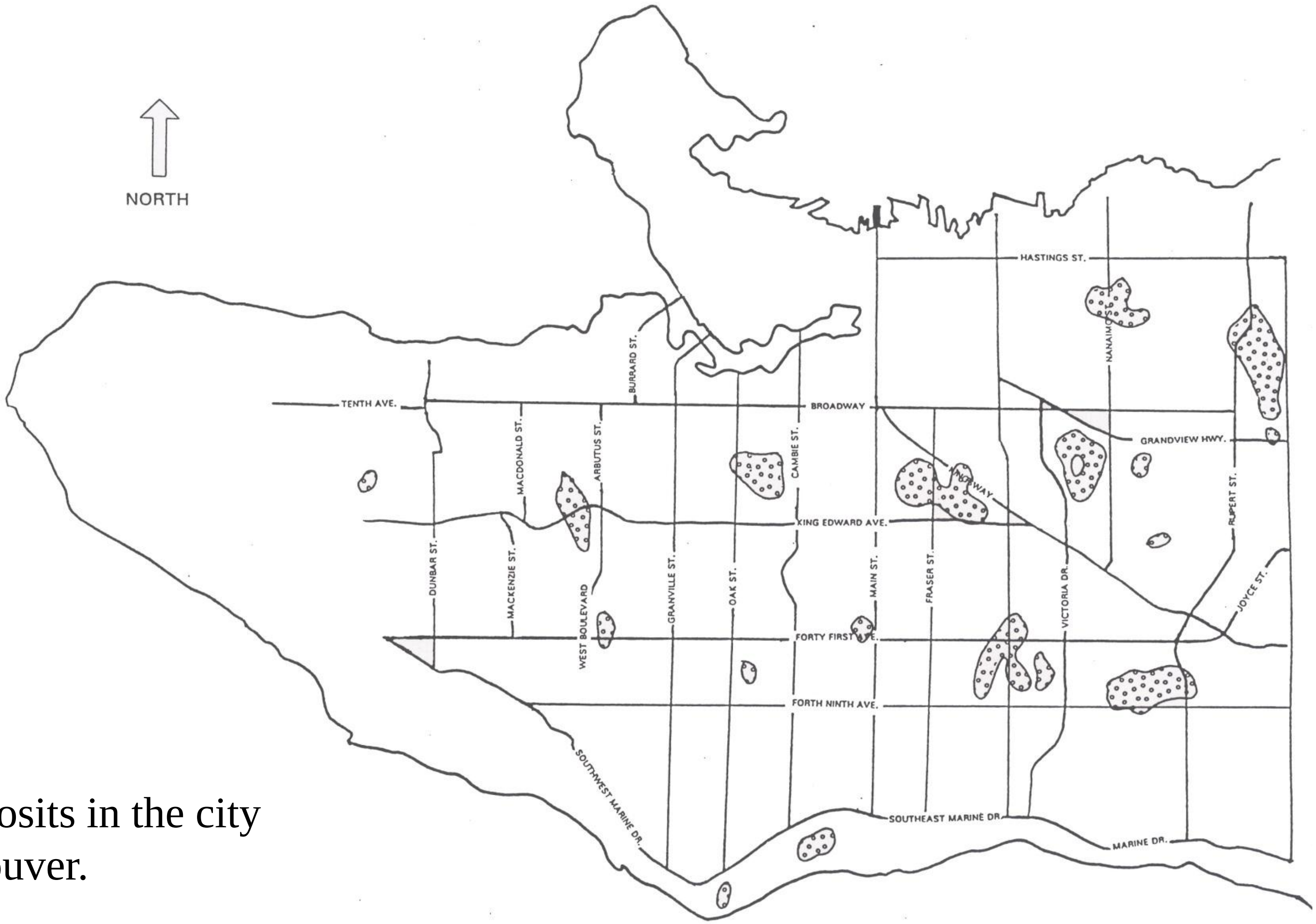


FIGURE 7.29 A drainage scheme for a footing.

Weak and Compressible Soils

- Typical soils
  - Soft clays, highly organic soils
- Typical locations
  - Near river mouths, perimeter of bays, beneath wetlands
- Typical problems
  - Bearing capacity and settlement problems
  - Loose saturated sands are liquefiable or suffer large movements under seismic loading
  - Areas subject to flooding

Peat Deposits in Vancouver



Peat deposits in the city of Vancouver.

Groundwater Control During Construction

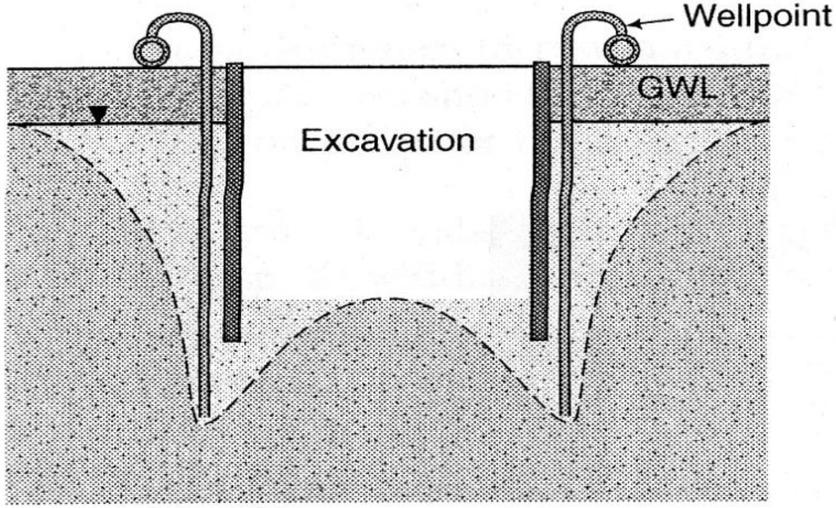


FIGURE 7.31 Dewatering a site using wellpoints.

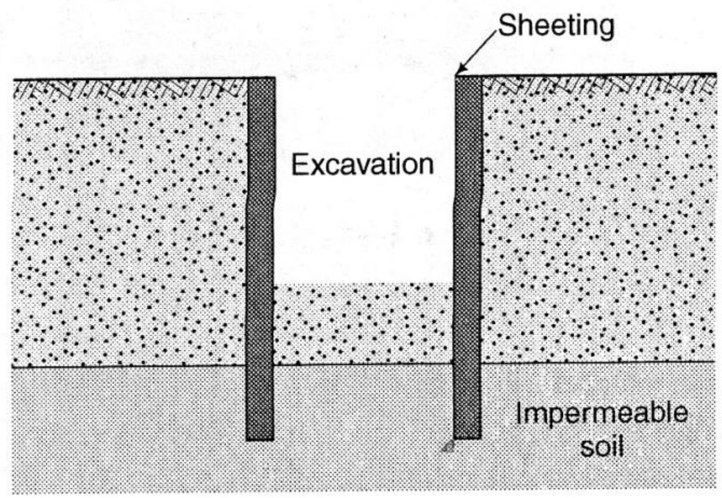


FIGURE 7.32 Sheet piles placed into an impermeable layer for dewatering a foundation.

Seismic Design Considerations

- Shaking intensity
- Ground motion amplification
- Liquefaction
- Loss of bearing capacity
- Post-cyclic settlements (differential settlements)
- Buoyancy

Liquefaction

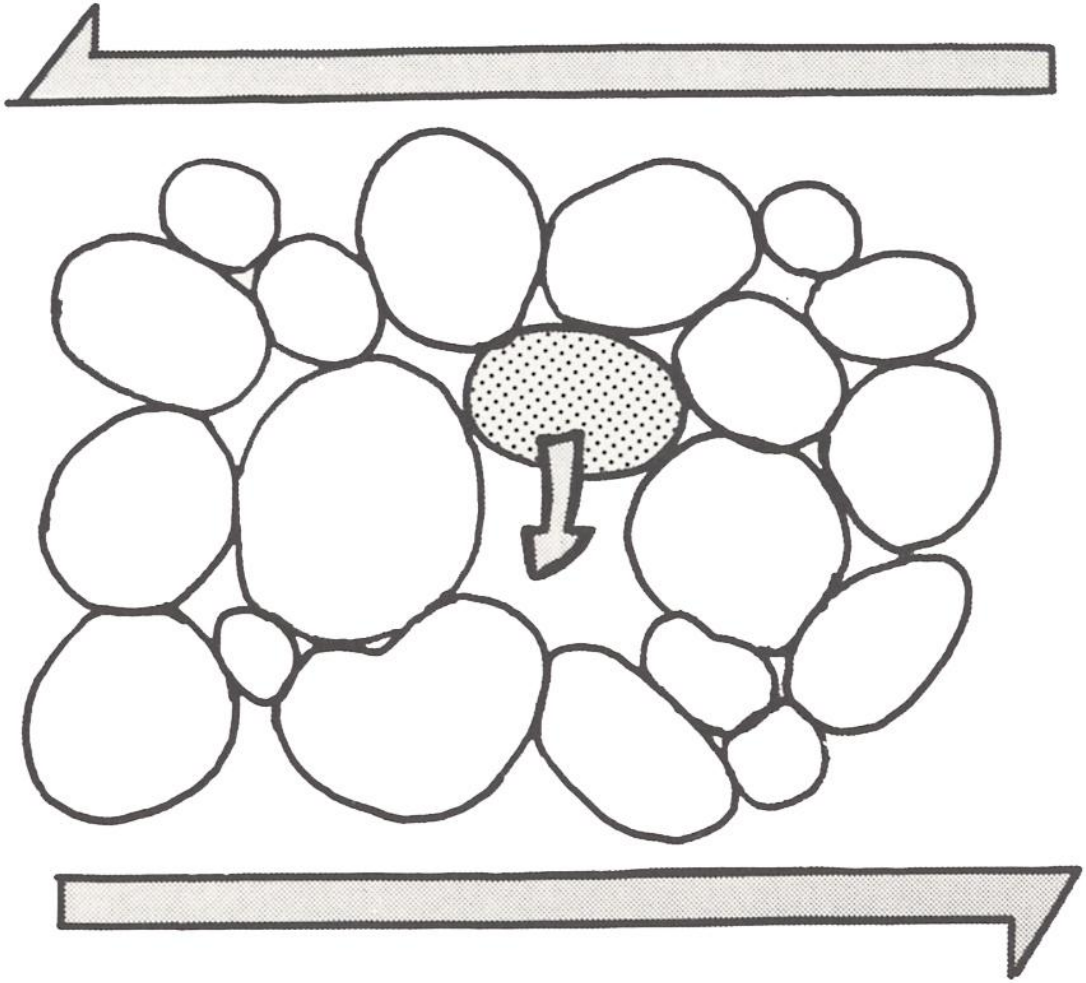
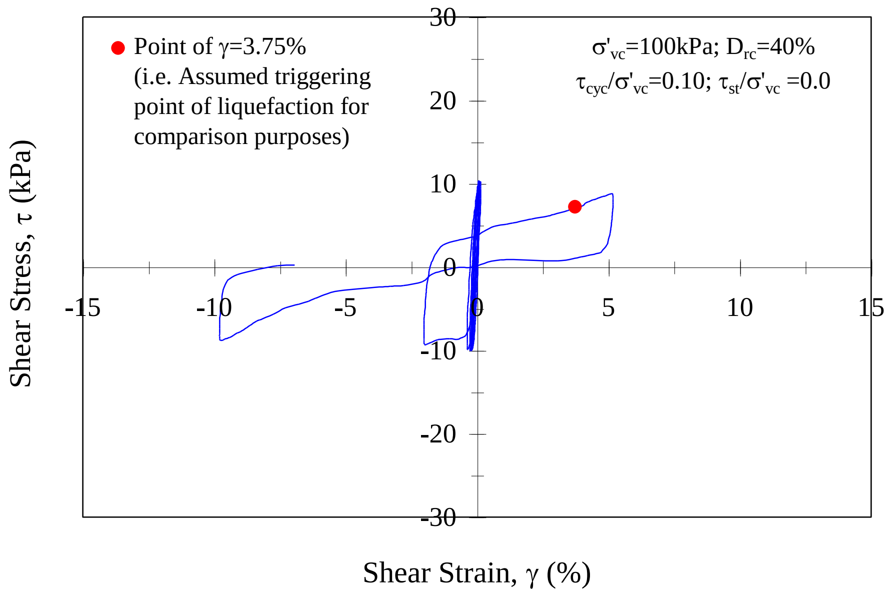


Figure 1

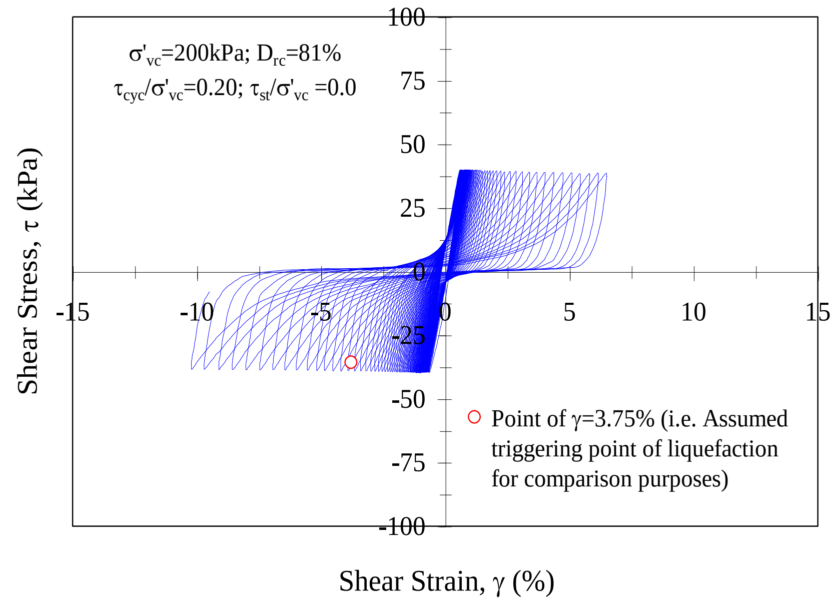
Sketch of a packet of water-saturated sand grains illustrating the process of liquefaction. Shear deformations (indicated by large arrows) induced by earthquake shaking distort the granular structure causing loosely packed groups to collapse as indicated by the curved arrow (Youd, 1992)

Laboratory Stress Strain Response
Fraser River Sand

Wijewickreme et al. (2005), Can Geotech. J. Vol. 42(2)



Loose Sand



Dense Sand

Typical Conditions for Liquefaction

- Cohesionless soil $s = \sigma' \tan \phi'$
- Loose
- Saturated
- Strong ground shaking
- Generally Undrained conditions

Liquefaction-Induced Ground Failures

- Flow slides
- Lateral spreads
- Ground oscillation
- Loss of bearing strength
- Settlement
- Increased lateral pressure on retaining walls

Flow Slides

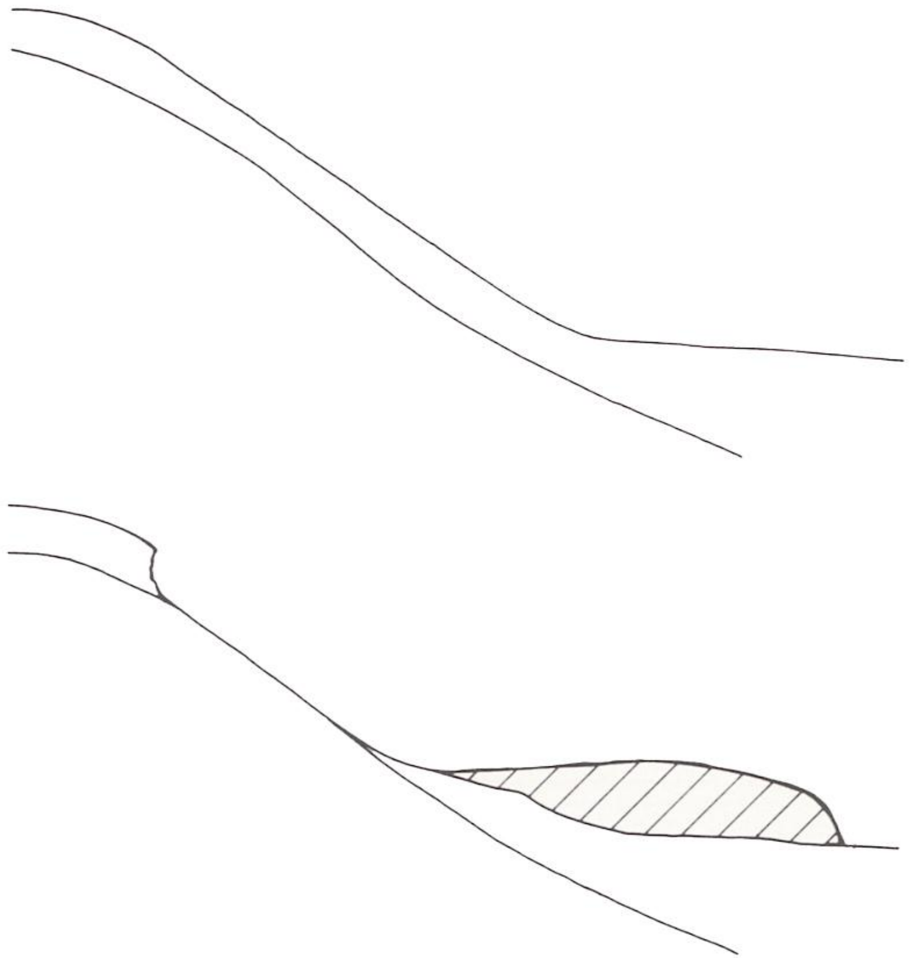


Figure 2

Diagram of a flow failure caused by liquefaction and loss of strength of soils lying on a steep slope. The strength loss creates instability and flow down the steep slope(Youd, 1992)

San Fernando, 1971

Lateral Spreads

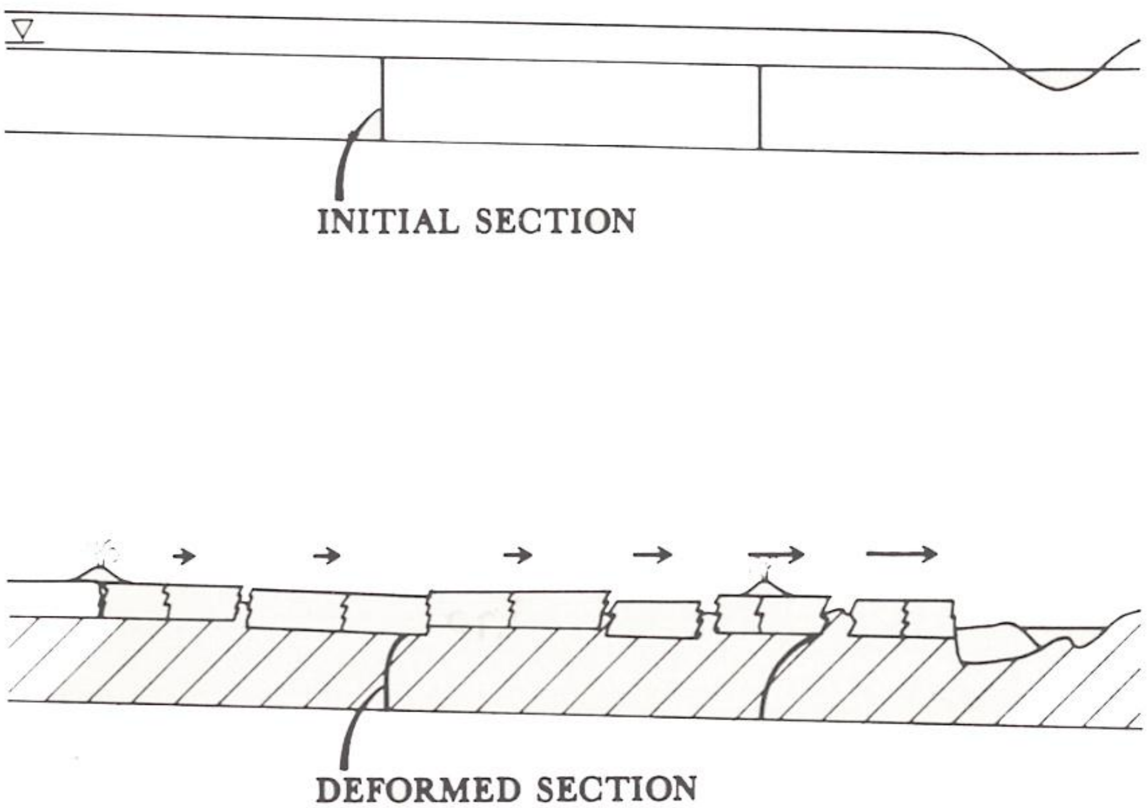


Figure 3

Diagram of a lateral spreading ground waves (Youd, 1992)

Liquefaction-induced Lateral Spreading

Loss of Bearing Capacity

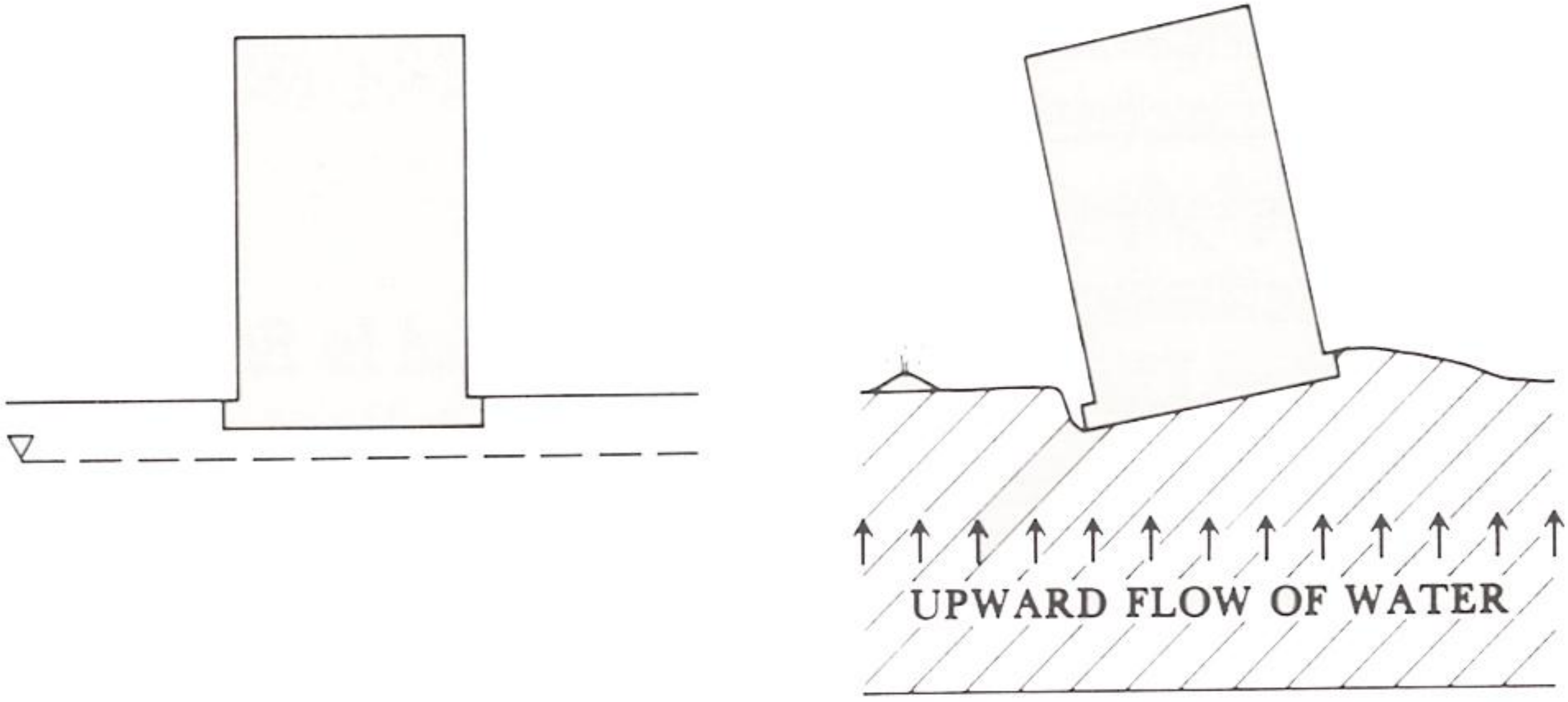
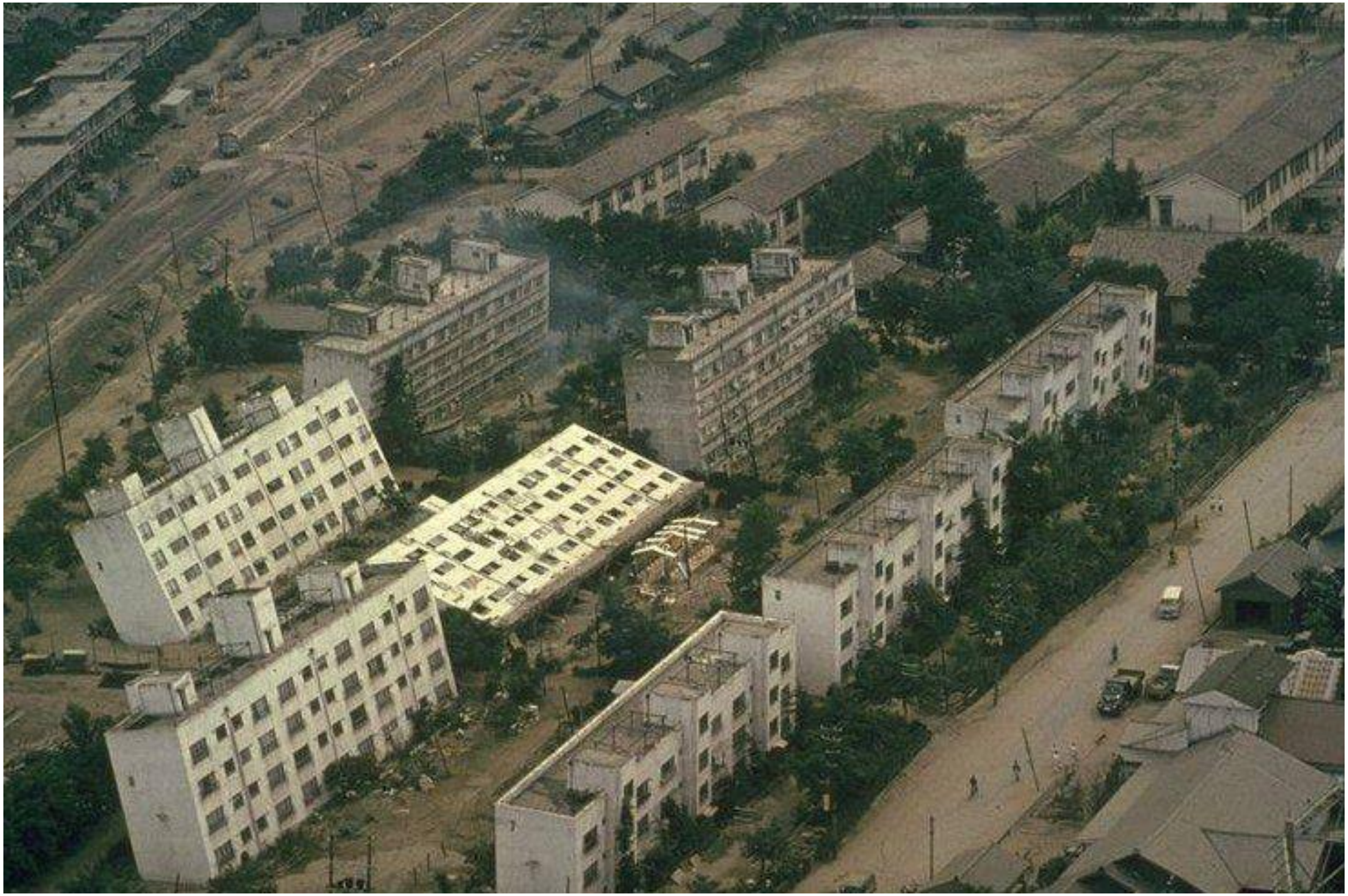


Figure 5

Diagram of structure tilted due to loss of bearing strength. Liquefaction weakens the soil reducing foundation support which allows heavy structures to settle and tip (Youd, 1992)

Niigata, Japan, 1964



Useful Websites

UBC Project on Earthquake Induced Damage Mitigation from Soil Liquefaction http://www.civil.ubc.ca/liquefaction/

Pacific Geoscience Centre, Victoria

http://www.pgc.nrcan.gc.ca/seismo/table.htm

University of Washington Liquefaction Site

http://www.ce.washington.edu/~liquefaction/html/main.html

Miscellaneous Considerations

- Frost protection
  (see section 7.12.3 Budhu text)
- Expansive soils
  (see section 7.12.4 Budhu text)
- Adjacent structures
- Global stability

Module 7

Design of Shallow Foundations

Overall Learning Objectives

- Ultimate Limit States – estimation of limit load by theoretical and empirical means (Book 2 pp. 200 – 224, 234-235)

- Serviceability Limit State – estimation of settlements of foundations on clay (Book 2 pp.224-234)

- Serviceability Limit State – estimation of settlements of foundations on sand (Book 2 pp. 235-240)

- Design Issues and Procedures (Book 2 pp. 251 – 270)