Version SKH/3

EGT3
ENGINEERING TRIPOS PART IIB

Thursday 1 May 2025 14.00 to 15.40

Module 4D5

FOUNDATION ENGINEERING

*Answer not more than **three** questions.*

All questions carry the same number of marks.

*The **approximate** percentage of marks allocated to each part of a question is indicated in the right margin.*

*Write your candidate number **not** your name on the cover sheet.*

STATIONERY REQUIREMENTS
Single-sided script paper

SPECIAL REQUIREMENTS TO BE SUPPLIED FOR THIS EXAM
CUED approved calculator allowed
Attachment: 4D5 Foundation Engineering Databook (21 pages)
Engineering Data Book

10 minutes reading time is allowed for this paper at the start of the exam.

You may not start to read the questions printed on the subsequent pages of this question paper until instructed to do so.

You may not remove any stationery from the Examination Room.

1 (a) Piles may be installed using either displacement or non-displacement methods.

(i) Describe the construction process for each of these methods for an onshore pile. [20%]

(ii) What are the benefits and disadvantages of the two construction methods? [20%]

(b) Describe the stress changes that occur for a soil element on the pile centreline during the installation of a driven pile in clay. How does the soil behaviour lead to the change of slope seen in the equation in the API design code for α given below?

$$\alpha = 0.5 \times \text{Max}\left(\left(\frac{\sigma_v'}{s_u}\right)^{0.5}, \left(\frac{\sigma_v'}{s_u}\right)^{0.25}\right)$$

[20%]

(c) A closed-ended driven pile with a diameter of 0.5 m is to be constructed in a clay with a uniform undrained strength s_u of 40 kPa and a bulk unit weight of 20 kN m^{-3}. Assuming the water table to be at the surface, what minimum length of pile is required to carry a vertical load of 350 kN? [20%]

(d) Ignoring structural failure of the pile material, what horizontal load could the pile carry at the ground surface if the pile was 10 m long? [20%]

2 (a) Foundations may be considered to have failed due to either the ultimate limit state or servicability limit state. Which criterion typically dominates design for onshore and offshore design and why? [20%]

(b) Why are factors of safety on ultimate limit state typically used in design? What are the dangers of using these in unfamiliar soil types? [20%]

(c) Derive from first principles an equation for the shaft stiffness of a rigid pile with radius R and length L in a uniform elastic soil with a shear stiffness G. [20%]

(d) A 20 m long tubular steel pile with an outer diameter of 1 m and a wall thickness of 20 mm is constructed in a clay layer whose stiffness increases linearly from $G = 20$ MPa at the surface to 50 MPa at the base of the pile with a Poisson's ratio $\nu = 0.4$. Assuming $\zeta = 4$, calculate the stiffness of the pile to vertical loads. [25%]

(e) Why does increasing the length of a pile not necessarily increase its stiffness? [15%]

3 Figure 1 shows an excavation with a depth $h = 7$ m and a width $B = 10$ m carried out to construct a cut-and-cover metro tunnel. The excavation is supported by a steel sheet pile wall with one level of temporary props at the top. The soil profile consists of a layer of sand with voids ratio $e = 0.9$ and a thickness of 7 m, followed by 2 m of normally consolidated clay and then by a second layer of fine, denser sand ($e = 0.7$), extending to significant depth. Both sands have specific gravity $G_S = 2.7$. The groundwater level is 6 m below ground surface. The characteristic values of the mechanical properties of the soils are indicated in Fig. 1.

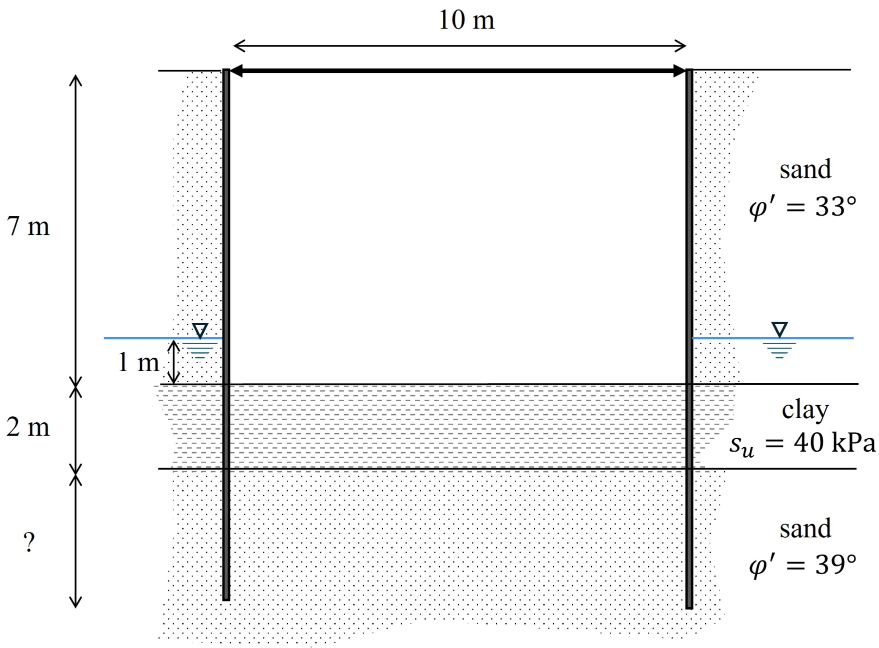


Fig. 1

(a) Compute the unit weight of the two layers of sand in dry and saturated conditions. [10%]

(b) Assuming that the mobilised friction at the interface between the wall and the soil on the passive side is $\delta = \varphi'/3$, compute the design active and passive lateral earth pressure using partial safety factor $\gamma_{\varphi'} = 1.25$ and Rankine's and Lancellotta's static solutions, respectively:

$$K_A = \frac{1 - \sin\varphi'}{1 + \sin\varphi'}$$

$$K_P = \frac{\cos\delta}{1 - \sin\varphi'}\left[\cos\delta + \sqrt{(\sin\varphi')^2 - (\sin\delta)^2}\right] e^{2\Theta\tan\varphi'}$$

where:

$$2\Theta = \sin^{-1}\left(\frac{\sin\delta}{\sin\varphi'}\right) + \delta$$

[25%]

(c) Sketch the short-term pore water pressure distribution in the sand layers on either side of the wall. Is there any potential base heave instability problem? Discuss. [25%]

(d) Using partial safety factors $\gamma_{\varphi'} = 1.25$ and $\gamma_{s_u} = 1.4$, compute the depth of embedment required to ensure stability of the wall in the short term. [40%]

4 A metro tunnel with a diameter of 5 m is constructed in a uniform layer of overconsolidated clay with its axis at a depth of 20 m. The clay has a unit weight of 20 kN m^{-3}, a coefficient of earth pressure at rest $K_0 = 1$, an undrained shear strength $s_u = 80$ kPa constant with depth, and a shear modulus of 10 MN m^{-2}. The measured radial ground movement at the tunnel boundary is 30 mm.

(a) By assuming the tunnel construction to be an axisymmetric contracting cavity under undrained conditions, estimate the average radial stress imposed on the tunnel lining, which is assumed to be smooth. [40%]

(b) At one location, the tunnel passes beneath a deep foundation supporting a sensitive building. The base of the foundation is 4 m above the crown of the tunnel. Ignoring any effects of the loading on the foundation, and assuming the same average radial stress imposed on the lining as in part (a), estimate the ground settlement at the level of the foundation. [20%]

(c) In the elastic zone of the soil, at any radius r, the following expressions apply:

$$\sigma_r = \sigma_0 - G\delta A/\pi r^2$$

$$\sigma_\theta = \sigma_0 + G\delta A/\pi r^2$$

where σ_r and σ_θ are the radial and hoop stress respectively, σ_0 is the original *in-situ* total stress in the ground, G is the shear modulus of the soil, and δA is the contraction of the cavity (expressed as a change in its cross-sectional area, A). By considering the radius of the elastic/plastic boundary, and making the same assumptions as for parts (a) and (b), calculate the distance above the tunnel crown at which the foundation would have to be located to ensure that it remained above the plastic zone associated with tunnel construction. [40%]

END OF PAPER

Cambridge University Engineering Department

Supplementary Databook

Module 4D5: Foundation Engineering

SKH. January 2024

Section 1: Empirical correlations for geotechnical data

1.1 Undrained shear strength of clays (s_u)

$\left(\frac{s_u}{\sigma'_v}\right)_{nc} \approx 0.11 + 0.37\, I_p$ for normally consolidated clay, where I_p is the plasticity index

$\frac{s_u}{\sigma'_v} \approx \left(\frac{s_u}{\sigma'_v}\right)_{nc} n^{\Lambda}$ where $n = \sigma'_{v,c} / \sigma'_v$ is overconsolidation ratio; $\Lambda \approx 0.8$

$s_u = \frac{(q_{penetrometer} - \sigma_v)}{N_{penetrometer}}$ from q at tip load cell, where $N_{cone} \approx 14 \pm 2$; $N_{T\text{-}bar} \approx 12 \pm 2$

$s_u \approx 4.5 N_{60}$ kPa from SPT blow-count N_{60} in Standard Penetration Test

1.2 Drained shear strength of sands (friction and dilatancy)

definition of relative dilatancy $I_R = I_D\, I_C - 1$

definition of relative density $I_D = (e_{max} - e)(e_{max} - e_{min})$

SPT blow-count correlation $I_D \approx [N_{60} / (20 + 0.2\, \sigma'_v \text{ kPa})]^{0.5}$

definition of relative crushability $I_C = \ln(\sigma_c / p')$

aggregate crushing stress $\sigma_c \approx$ shelly carbonate sand 5 000 kPa

quartz sand 20 000 kPa

quartz silt 80 000 kPa

CPT correlation (q_{cone}, σ'_v in kPa) $I_D \approx 0.27(\ln q_{cone} - 0.5 \ln \sigma'_v) - 1.29 \pm 0.15$ (higher, σ_c lower)

peak friction correlation $(\phi_{max} - \phi_{crit}) \approx 0.8\, \psi_{max} \approx 5° \times I_R$ in plane strain

$(\phi_{max} - \phi_{crit}) \approx 3° \times I_R$ in axisymmetric strain

peak dilatancy rate $(-\delta\varepsilon_v / \delta\varepsilon_1)_{max} \approx 0.3 \times I_R$ in all conditions

critical state friction angle $\phi_{crit} \approx 32°$ (uniform, rounded) → 40° (well-graded, angular)

1.3 Stiffness of clays

initial linear elastic shear stiffness $G_0 = \frac{B}{(1+e)^{2.4}}(p')^{0.5}$ with G_0, p' in kPa

$B \approx 20\,000$

normalised secant shear stiffness $\frac{G}{G_0} \approx \frac{1}{1+\left(\frac{\gamma}{\gamma_{ref}}\right)^a}$

hyperbolic curvature parameter $a \approx 0.68$ for $n < 1.5$, $a \approx 0.77$ for $n > 1.5$

reference shear strain $\gamma_{ref} \approx A\, w_L 10^{-3}$ e.g. writing liquid limit 40% as $w_L = 0.4$

$A \approx 1.35$ for $n < 1.5$; $A \approx 1.02$ for $n > 1.5$

mobilised shear strength $\frac{\tau_{mob}}{s_u} \approx \left(\frac{\gamma}{\gamma_u}\right)^{0.5}$ for $s_{mob} < s_u$

$\gamma_u \approx 0.01$ to 0.04 increasing with plasticity index I_p

1.4 Stiffness of sands

initial linear elastic shear stiffness $G_0 = \frac{B}{(1+e)^{3}}(p')^{0.5}$ with G_0, p' in kPa

$B \approx 57\,600$

normalised secant shear stiffness $\frac{G}{G_0} = \frac{1}{1+\left(\frac{\gamma-\gamma_e}{\gamma_{ref}}\right)^a}$

hyperbolic curvature parameter $a = U_c^{-0.075}$ e.g. $a = 0.9$ at uniformity coefficient $U_c = 4$

reference shear strain $\gamma_{ref} = U_c^{-0.3}\, p'\, 10^{-6} + 8\, e\, I_D\, 10^{-4}$

elastic limiting strain $\gamma_e = 0.012\,\gamma_{ref} + 2.10^{-6}$

Section 2: Plasticity theory

This section is common with the Soil Mechanics Databook supporting modules 3D1 and 3D2. Undrained shear strength ('cohesion' in a Tresca material) is denoted by s_u rather than c_u.

2.1 Plasticity: Tresca material, $\tau_{max} = s_u$

Limiting stresses

Tresca $\quad |\sigma_1 - \sigma_3| = q_u = 2s_u$

von Mises $\quad (\sigma_1 - p)^2 + (\sigma_2 - p)^2 + (\sigma_3 - p)^2 = \frac{2}{3} q_u^2 = 2s_u^2$

q_u= undrained triaxial compression strength; s_u= undrained plane shear strength.

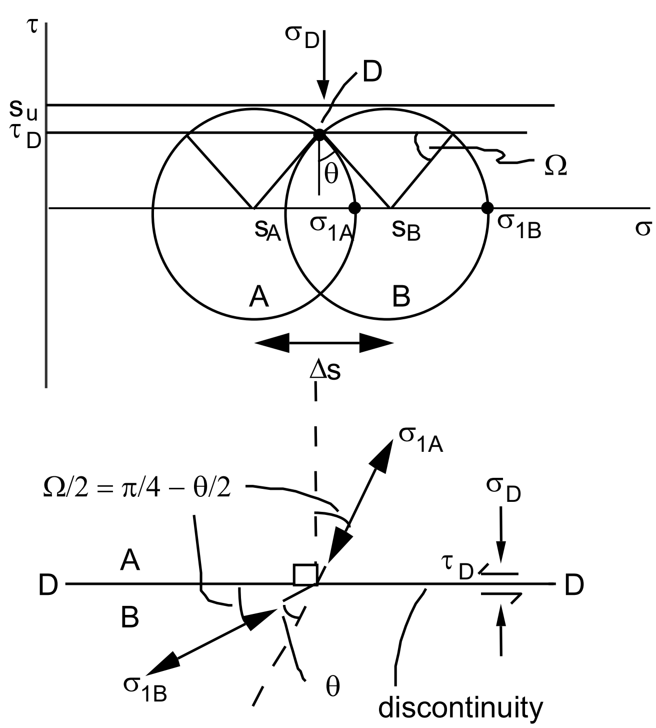
Dissipation per unit volume in plane strain deformation following either Tresca or von Mises,

$$\delta D = s_u \delta \varepsilon_\gamma$$

For a relative displacement x across a slip surface of area A mobilising shear strength s_u , this becomes

$$D = A s_u x$$

2.2 Stress conditions across a discontinuity:



Rotation of major principal stress

$$\theta = \pi/2 - \Omega$$

$$s_B - s_A = \Delta s = 2s_u \sin\theta$$
$$\sigma_{1B} - \sigma_{1A} = 2s_u \sin\theta$$

In limit with $\theta \rightarrow 0$

$$ds = 2s_u\, d\theta$$

Useful example:

$$\theta = 30^\circ$$

$$\sigma_{1B} - \sigma_{1A} = s_u$$

$$\tau_D / s_u = 0.87$$

σ_{1A} = major principal stress in zone A

σ_{1B} = major principal stress in zone B

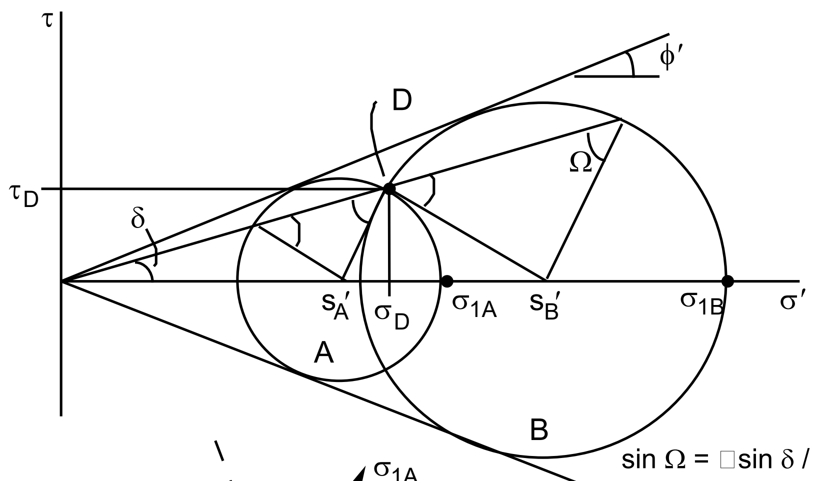
2.3 Plasticity: Coulomb material $(\tau/\sigma')_{max} = \tan\phi$

Limiting stresses

$$\sin\phi = (\sigma'_{1f} - \sigma'_{3f})/(\sigma'_{1f} + \sigma'_{3f}) = (\sigma_{1f} - \sigma_{3f})/(\sigma_{1f} + \sigma_{3f} - 2u)$$

where σ'_{1f} and σ'_{3f} are the major and minor principal effective stresses at failure, σ_{1f} and σ_{3f} are the major and minor principal total stresses at failure, and u is the pore pressure.

2.4 Stress conditions across a discontinuity



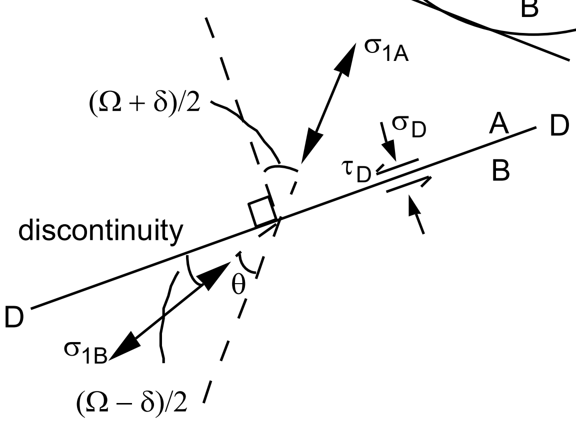
Rotation of major principal stress

$\theta = \pi/2 - \Omega$

σ_{1A} = major principal stress in zone A

σ_{1B} = major principal stress in zone B

$\tan\delta = \tau_D / \sigma'_D$



$\sin\Omega = \sin\delta / \sin\phi'$

$s'_B/s'_A = \sin(\Omega + \delta) / \sin(\Omega - \delta)$

In limit, $d\theta \to 0$ and $\delta \to \phi'$

$ds' = 2s'.\, d\theta \tan\phi'$

Integration gives $s'_B/s'_A = \exp(2\theta \tan\phi')$

Section 3: Bearing capacity of shallow foundations

3.1 Tresca soil, with undrained strength s_u

3.1.1 Vertical loading

The vertical bearing capacity, q_f, of a shallow foundation for undrained loading (Tresca soil) is:

$$\frac{V_{ult}}{A} = q_f = s_c d_c N_c s_u + \gamma h$$

V_{ult} and A are the ultimate vertical load and the foundation area, respectively. h is the embedment of the foundation base and γ (or γ') is the appropriate density of the overburden.

The exact bearing capacity factor N_c for a plane strain surface foundation (zero embedment) on uniform soil is:

$N_c = 2 + \pi$ (Prandtl, 1921)

Shape correction factor:
For a rectangular footing of length L and breadth B (Eurocode 7):

$s_c = 1 + 0.2 B / L$

The exact solution for a rough circular foundation (B/L=1) is $q_f = 6.05 s_u$, hence $s_c = 1.18 \sim 0.2$.

Embedment correction factor:
A fit to Skempton's (1951) embedment correction factors, for an embedment of h, is:

$d_c = 1 + 0.33 \tan^{-1} (h/D)$ (or h/B for a strip or rectangular foundation)

3.1.2 Combined V-H loading

A curve fit to Green's lower bound plasticity solution for V-H loading is:

If $V/V_{ult} > 0.5$: $\frac{V}{V_{ult}} = \frac{1}{2} + \frac{1}{2}\sqrt{1 - \frac{H}{H_{ult}}}$ or $\frac{H}{H_{ult}} = 1 - \left(2\frac{V}{V_{ult}} - 1\right)^2$

If $V/V_{ult} < 0.5$: $H = H_{ult} = B s_u$

3.1.3 Combined V-H-M loading

With lift-off: combined Green-Meyerhof ($V\rho_{ult}$ = bearing capacity of effective area B-e)

If $V/V\rho_{ult} < 0.5$: $\frac{H}{H_{ult}} = \left(1 - 2\frac{M}{VB}\right)$

Without lift-off: $\left(\frac{V}{V_{ult}}\right)^2 + \left[\frac{M}{M_{ult}}\left(1 - 0.3\frac{H}{H_{ult}}\right)\right]^2 + \left|\left(\frac{H}{H_{ult}}\right)^3\right| - 1 = 0$ (Taiebat & Carter 2000)

3.2 Frictional (Coulomb) soil, with friction angle ϕ

3.2.1 Vertical loading

The vertical bearing capacity, q_f, of a shallow foundation under drained loading (Coulomb soil) is:

$$\frac{V_{ult}}{A} = q_f = s_q N_q \sigma'_{v0} + s_\gamma N_\gamma \frac{\gamma' B}{2}$$

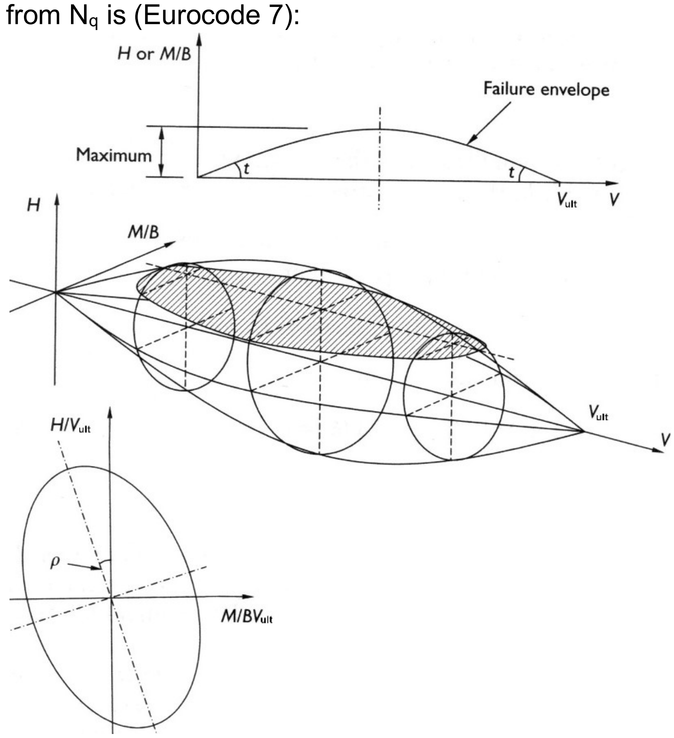
The bearing capacity factors N_q and N_γ account for the capacity arising from surcharge and self-weight of the foundation soil respectively. σ'_{v0} is the in situ effective stress acting at the level of the foundation base.

For a strip footing on weightless soil, the exact solution for N_q is:

$N_q = \tan^2(\pi/4 + \phi/2)\, e^{(\pi \tan \phi)}$ (Prandtl 1921)

An empirical relationship to estimate N_γ from N_q is (Eurocode 7):

$N_\gamma = 2 (N_q - 1) \tan \phi$



Curve fits to exact solutions for $N_\gamma = f(\phi)$ are (Davis & Booker 1971):

Rough base: $N_\gamma = 0.1054\, e^{9.6\phi}$

Smooth base: $N_\gamma = 0.0663\, e^{9.3\phi}$

Shape correction factors:

For a rectangular footing of length L and breadth B (Eurocode 7):

$s_q = 1 + (B \sin \phi) / L$
$s_\gamma = 1 - 0.3\, B / L$

For circular footings assume L = B.

3.2.2 Combined V-H loading

The Green/Sokolovski lower bound solution gives a V-H failure surface.

3.2.3 Combined V-H-M loading

(with lift-off- drained conditions- see failure surface shown above)

$$\left[\frac{H/V_{ult}}{t_h}\right]^2 + \left[\frac{M/BV_{ult}}{t_m}\right]^2 + \left[\frac{2C(M/BV_{ult})(H/V_{ult})}{t_h t_m}\right] = \left[\frac{V}{V_{ult}}\left(1 - \frac{V}{V_{ult}}\right)\right]^2$$

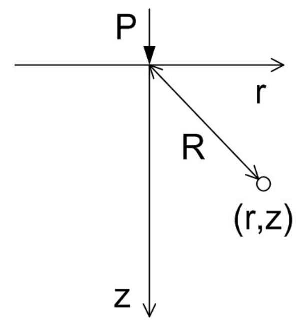
where $C = \tan\left(\frac{2\rho(t_h - t_m)(t_h + t_m)}{2 t_h t_m}\right)$ (Butterfield & Gottardi 1994)

Typically, t_h~0.5, t_m~0.4 and ρ~15°. t_h is the friction coefficient, H/V= $\tan \phi$, during sliding.

Section 4: Settlement of shallow foundations

4.1 Elastic stress distributions below point, strip and circular loads

Point loading (Boussinesq solution)



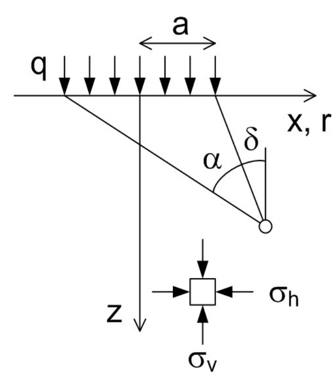
Vertical stress $\sigma_z = \dfrac{3Pz^3}{2\pi R^5}$

Radial stress $\sigma_r = \dfrac{P}{2\pi R^2}\left[\dfrac{3r^2 z}{R^3} - \dfrac{(1-2\nu)R}{R+z}\right]$

Tangential stress $\sigma_\theta = \dfrac{P(1-2\nu)}{2\pi R^2}\left[\dfrac{R}{R+z} - \dfrac{z}{R}\right]$

Shear stress $\tau_{rz} = \dfrac{3Prz^2}{2\pi R^5}$

Uniformly-loaded strip

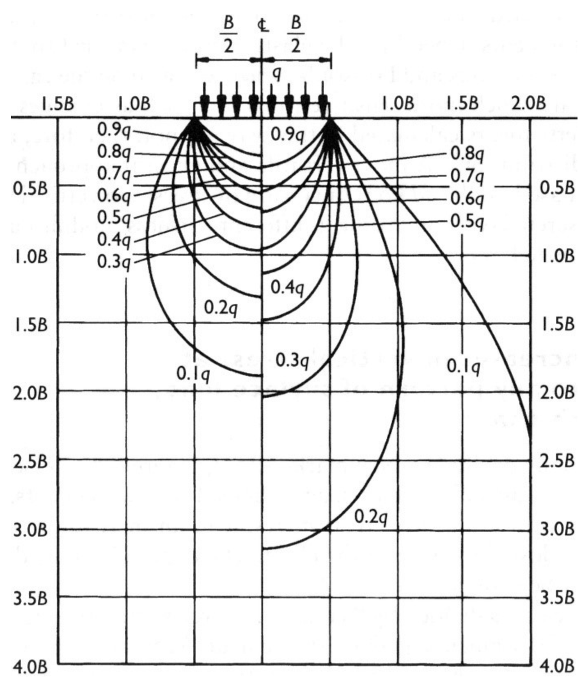


Vertical stress $\sigma_v = \dfrac{q}{\pi}\left[\alpha + \sin\alpha\cos(\alpha + 2\delta)\right]$

Horizontal stress $\sigma_h = \dfrac{q}{\pi}\left[\alpha - \sin\alpha\cos(\alpha + 2\delta)\right]$

Shear stress $\tau_{vh} = \dfrac{q}{\pi}\sin\alpha\sin(\alpha + 2\delta)$

Principal stresses



$$\sigma_1 = \frac{q}{\pi}(\alpha + \sin\alpha) \quad \sigma_3 = \frac{q}{\pi}(\alpha - \sin\alpha)$$

Uniformly-loaded circle

(on centerline, r=0)

Vertical stress

$$\sigma_v = q\left[1-\left(\frac{1}{1+(a/z)^2}\right)^{\frac{3}{2}}\right]$$

Horizontal stress

$$\sigma_h = \frac{q}{2}\left[(1+2\nu)-\frac{2(1+\nu)z}{(a^2+z^2)^{1/2}}+\frac{z^3}{(a^2+z^2)^{3/2}}\right]$$

Contours of vertical stress below uniformly-loaded circular (left) and strip footings (right)

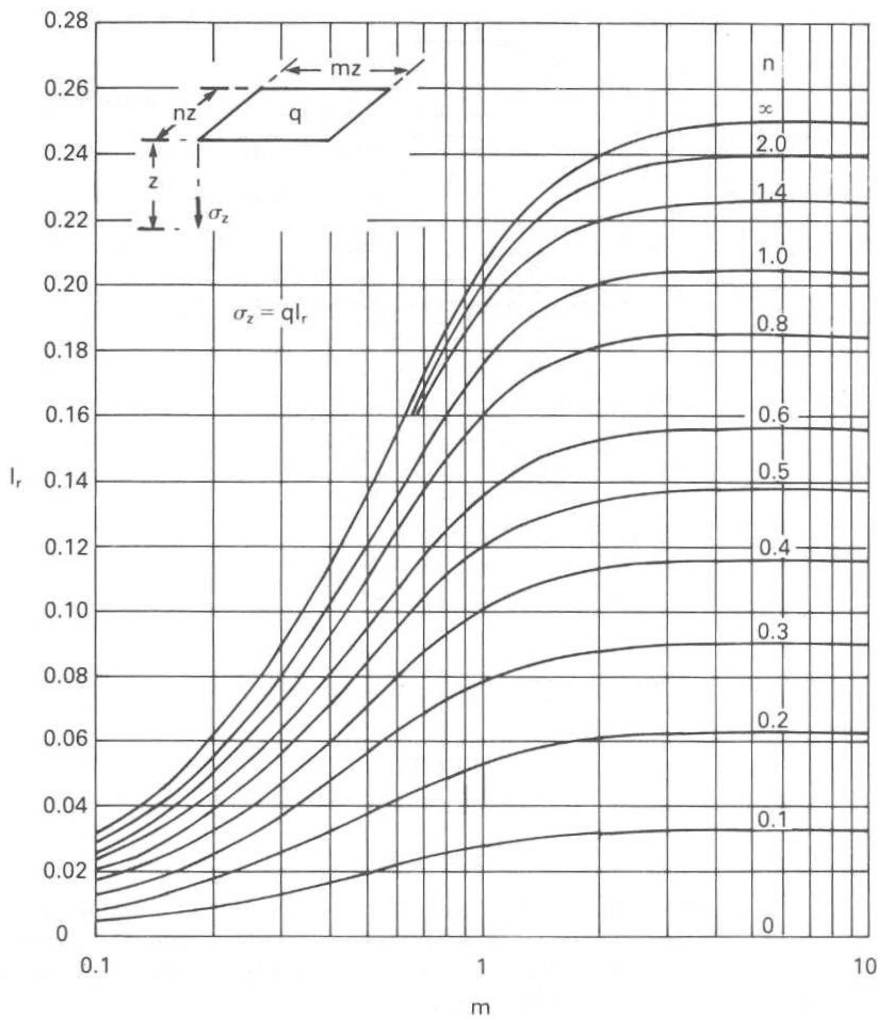
4.2 Elastic stress distribution below rectangular area

The vertical stress, σ_z, below the corner of a uniformly-loaded rectangle (L × B) is:

$$\sigma_z = I_r q$$

I_r is found from m (=L/z) and n (=B/z) using Fadum's chart or the expression below (L and B are interchangeable), which are from integration of Boussinesq's solution.

$$I_r = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right)+\tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right]$$



Influence factor, I_r, for vertical stress under the corner of a uniformly-loaded rectangular area (Fadum's chart)

4.3 Elastic solutions for surface settlement

4.3.1 Isotropic, homogeneous, elastic half-space (semi-infinite)

Point load (Boussinesq solution)

Settlement, w, at distance s: $w(s) = \frac{1}{2\pi}\frac{(1-\nu)}{G}\frac{P}{s}$

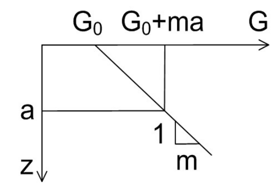
Circular area (radius a), uniform soil

Uniform load: central settlement: $w_o = \frac{(1-\nu)}{G}qa$

edge settlement: $w_e = \frac{2}{\pi}\frac{(1-\nu)}{G}qa$

Rigid punch: (q_{avg}= $V/\pi a^2$) $w_r = \frac{\pi}{4}\frac{(1-\nu)}{G}q_{avg}a$

Circular area, heterogeneous soil



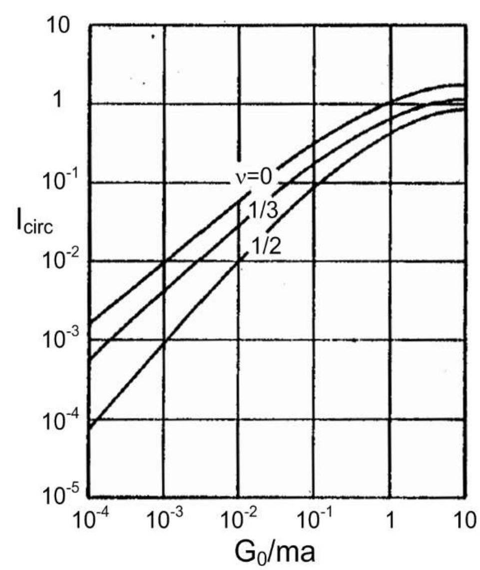
For G_0= 0, ν= 0.5:

w= q/2m under loaded area of any shape
w= 0 outside loaded area

For G_0> 0, central settlement:

$$w_o = \frac{qa}{2G_0}I_{circ}$$

For ν= 0.5, $w_o \approx \frac{qa}{2(G_0 + ma)}$



Rectangular area, uniform soil

Uniform load, corner settlement:

$$w_c = \frac{(1-\nu)}{G}\frac{qB}{2}I_{rect}$$

Where I_{rect} depends on the aspect ratio, L/B:

L/B	I_{rect}	L/B	I_{rect}	L/B	I_{rect}	L/B	I_{rect}
1	0.561	1.6	0.698	2.4	0.822	5	1.052
1.1	0.588	1.7	0.716	2.5	0.835	6	1.110
1.2	0.613	1.8	0.734	3	0.892	7	1.159
1.3	0.636	1.9	0.750	3.5	0.940	8	1.201
1.4	0.658	2	0.766	4	0.982	9	1.239
1.5	0.679	2.2	0.795	4.5	1.019	10	1.272

Rigid rectangle: $w_r = \frac{(1-\nu)}{G}\frac{q_{avg}\sqrt{BL}}{2}I_{rgd}$ where I_{rgd} varies from 0.9→0.7 for L/B = 1-10.

Note: $G = \frac{E}{2(1+\nu)}$ where ν= Poisson's ratio, E= Young's modulus.

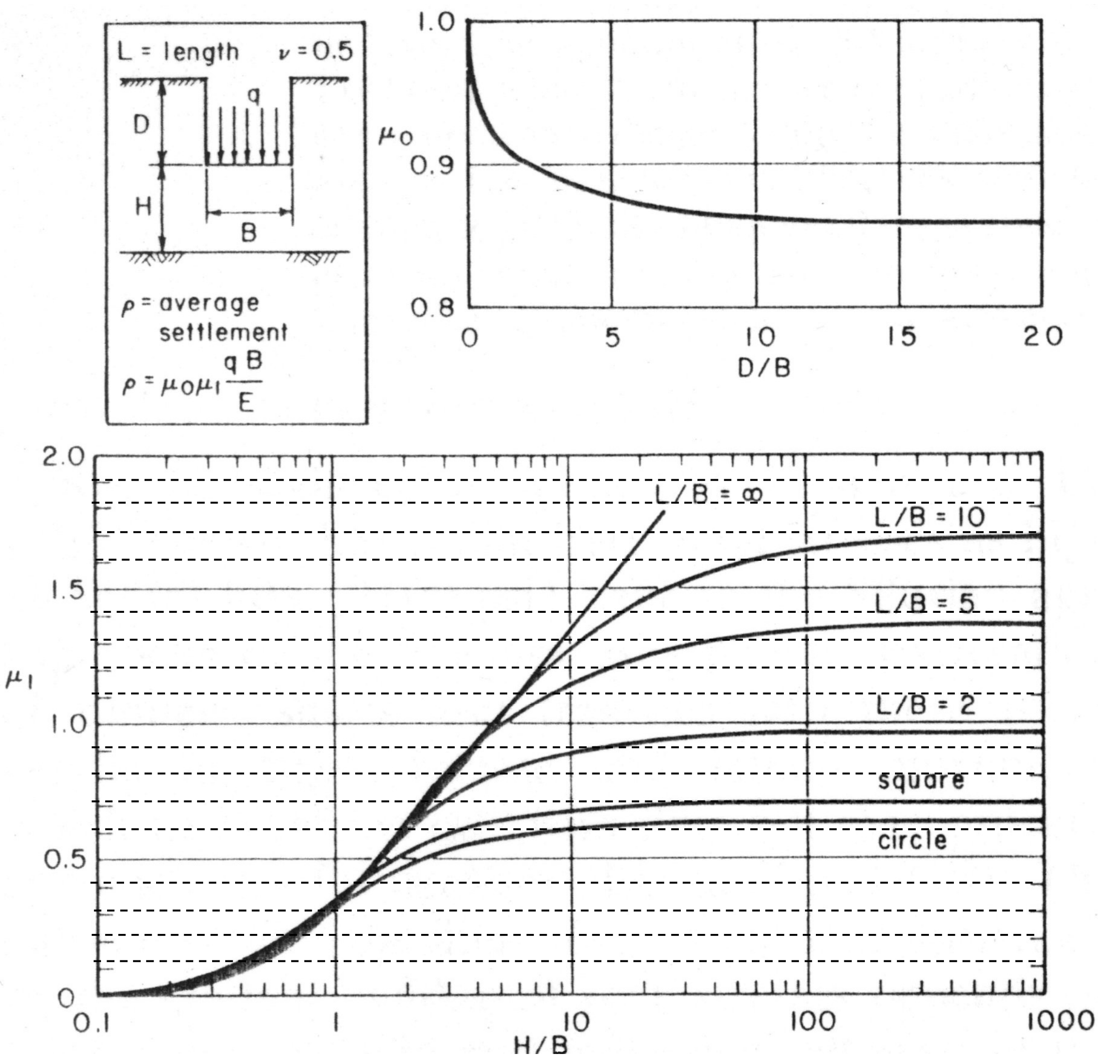
4.3.2 Isotropic, homogeneous, elastic finite space

Elastic layer of finite thickness

The mean settlement of a uniformly loaded foundation embedded in an elastic layer of finite thickness can be found using the charts below, for $\nu \sim 0.5$.

$$w_{avg} = \mu_0 \mu_1 \frac{qB}{E} \qquad\qquad E = 2G(1+\nu)$$

The influence factor μ_1 accounts for the finite layer thickness. The influence factor μ_0 accounts for the embedded depth.



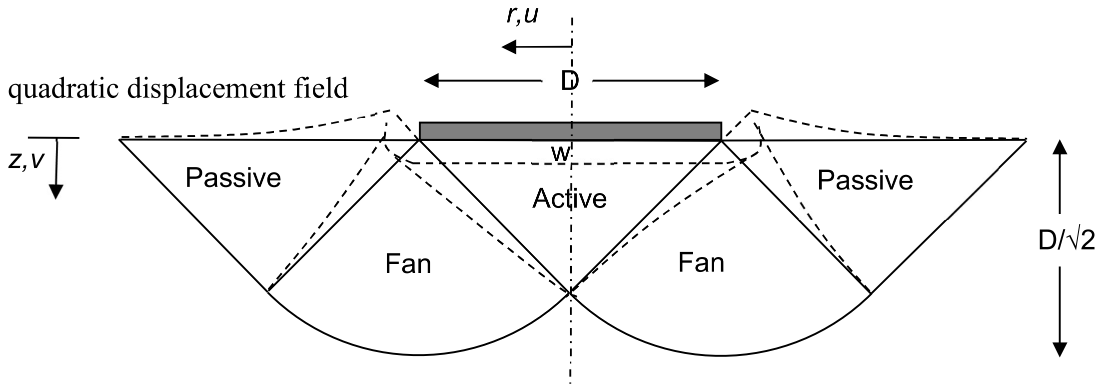
Average immediate settlement of a uniformly loaded finite thickness layer

Christian & Carrier (1978) Janbu, Bjerrum and Kjaernsli's chart reinterpreted. Canadian Geotechnical Journal (15) 123-128.

4.4 Mobilizable Strength Design (MSD) solutions

Rigid circular foundation on incompressible half-space

(Osman & Bolton, 2005)



Vertical bearing stress q

Average shear strain within deformation mechanism: $\gamma_{mob} = M_c\ w/D = 1.35\ w/D$

Average shear stress mobilized within mechanism: $\tau_{mob} = q / N_c = q / 5.9$

Representative depth to identify shear stress-strain behaviour: $z_{rep} = 0.3D$

If the representative soil test data fits: $\tau_{mob} = f(\gamma_{mob})$

Assume that the foundation load test data would fit: $(q/5.9) = f(1.35\ w/D)$

NB: this will underestimate w/D as $q \rightarrow 5.9\ s_u$, due to local strain concentrations

Horizontal or Moment loading

See Osman et al. (2007) Geotechnique 57 (9) 729-737

Section 5: Bearing capacity of deep foundations

5.1 Axial capacity: API (2000) design method for driven piles

5.1.1 Sand

Unit shaft resistance: $\tau_{sf} = \sigma'_{hf} \tan\delta = K\sigma'_{v0} \tan\delta \leq \tau_{s,lim}$

Closed-ended piles: K = 1
Open-ended piles: K = 0.8

Unit base resistance: $q_b = N_q\, \sigma'_{vo} < q_{b,limit}$

Soil category	Soil density	Soil type	Soil-pile friction angle, δ (°)	Limiting value $\tau_{s,lim}$ (kPa)	Bearing capacity factor, N_q	***Limiting value, $q_{b,lim}$ (MPa)***
1	Very loose Loose Medium	Sand Sand-silt Silt	15	50	8	1.9
2	Loose Medium Dense	Sand Sand-silt Silt	20	75	12	2.9
3	Medium Dense	Sand Sand-silt	25	85	20	4.8
4	Dense Very dense	Sand Sand-silt	30	100	40	9.6
5	Dense Very dense	Gravel Sand	35	115	50	12

API (2000) recommendations for driven pile capacity in sand

5.1.2 Clay

American Petroleum Institute (API) (2000) guidelines for driven piles in clay.

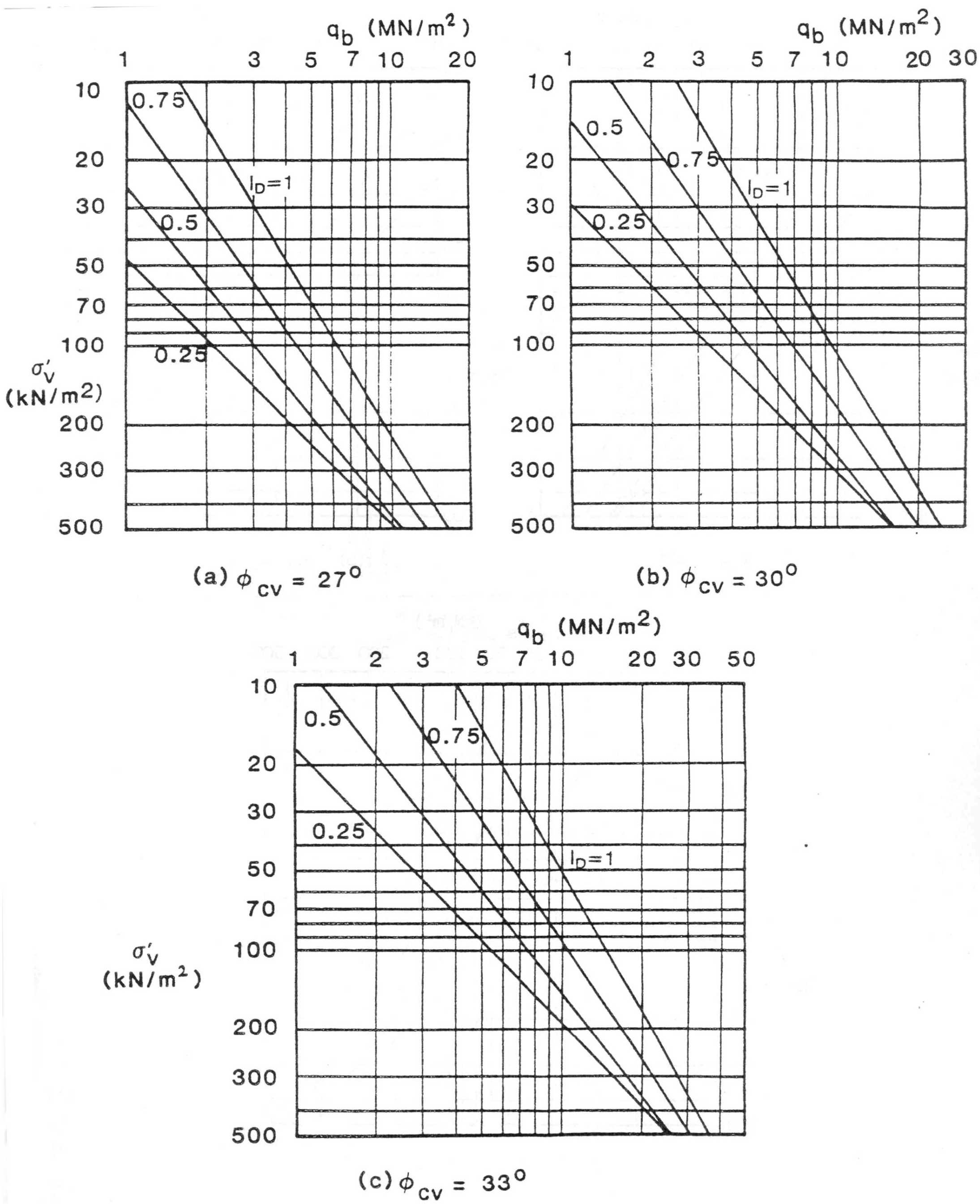
***Unit shaft resistance*:** $$\alpha = \frac{\tau_s}{s_u} = 0.5 \cdot \mathrm{Max}\left[\left(\frac{\sigma'_{vo}}{s_u}\right)^{0.5}, \left(\frac{\sigma'_{vo}}{s_u}\right)^{0.25}\right]$$

It is assumed that equal shaft resistance acts inside and outside open-ended piles.

***Unit base resistance*:** $q_b = N_c\, s_u$ $N_c = 9$.

5.2 Axial capacity: base resistance in sand using Bolton's stress dilatancy

Unit base resistance, q_b, is expressed as a function of relative density, I_D, constant volume (critical state) friction angle, ϕ_{cv}, and in situ vertical effective stress, σ'_v.



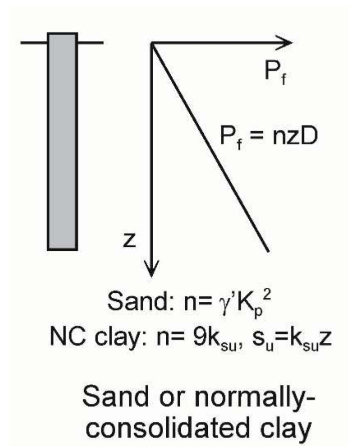
Design charts for base resistance in sand
(Randolph 1985, Fleming et al 1992)

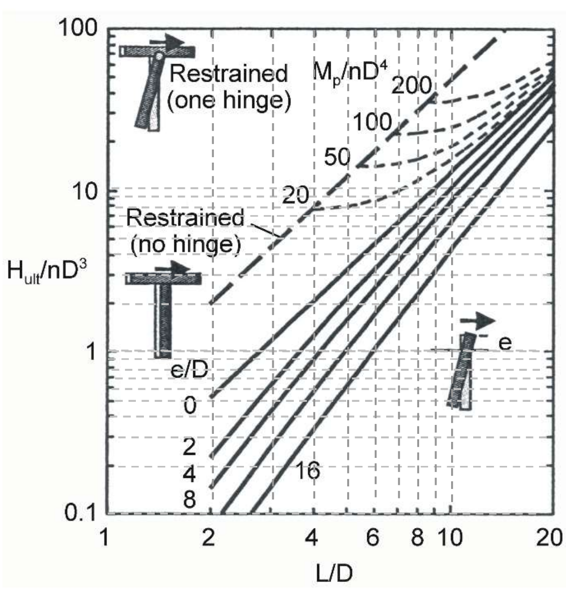
5.3 Lateral capacity: linearly increasing lateral resistance with depth

Lateral soil resistance (force per unit length), $P_u = nzD$

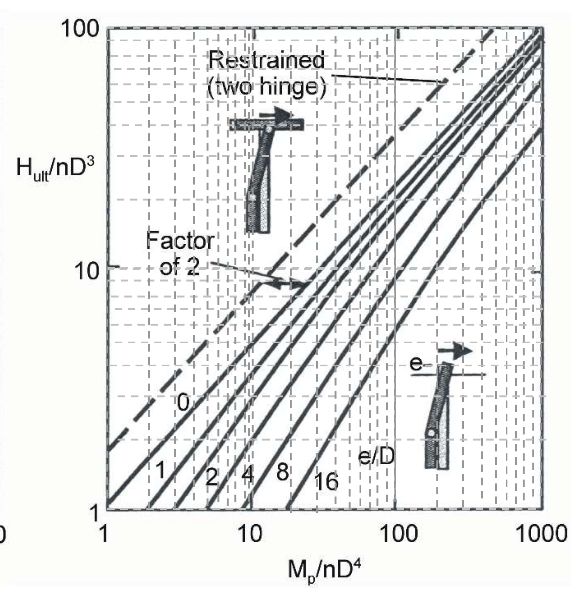
In sand, $n = \gamma' K_p^2$
In normally consolidated clay with strength gradient k; $s_u = kz$; n=9k

H_{ult} ultimate horizontal load on pile
M_p plastic moment capacity of pile
D pile diameter
L pile length
e load level above pile head
(=M/H for H-M pile head loading)
γ' effective unit weight
K_p passive earth pressure coefficient,
$K_p = (1+ \sin \phi)/(1- \sin \phi)$





Short pile failure mechanism



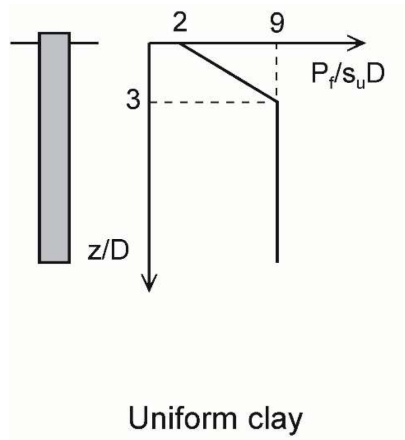
Long pile failure mechanism

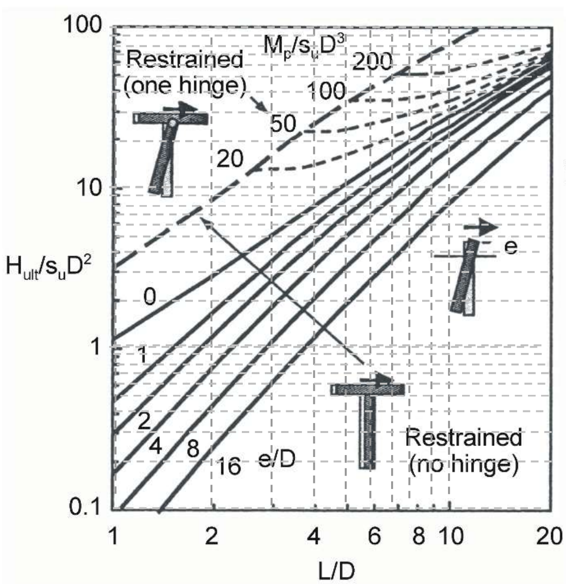
Lateral pile capacity
(linearly increasing lateral resistance with depth)

5.4 Lateral capacity: uniform clay

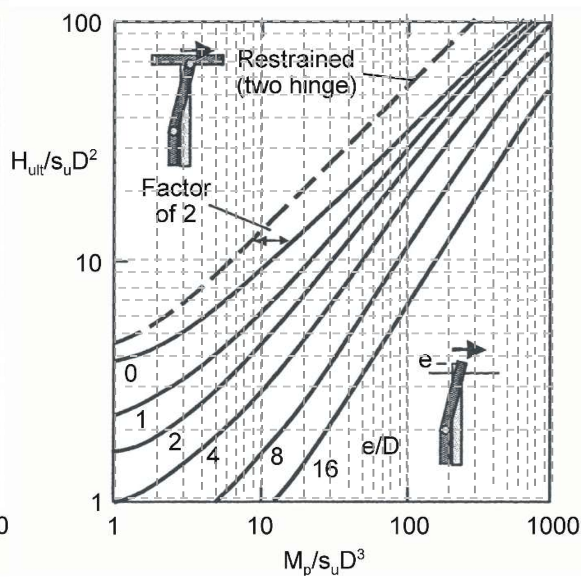
Lateral soil resistance (force per unit length), P_u, increases from $2s_uD$ at surface to $9s_uD$ at 3D depth then remains constant.

H_{ult}	ultimate horizontal load on pile
M_p	plastic moment capacity of pile
D	pile diameter
L	pile length
e	load level above pile head (=M/H for H-M pile head loading)
s_u	undrained shear strength





Short pile failure mechanism



Long pile failure mechanism

Lateral pile capacity
(uniform clay lateral resistance profile)

Section 6: Settlement of deep foundations

6.1 Settlement of a rigid pile

Shaft response:

Equilibrium:

$$\tau = \tau_s \frac{R}{r}$$

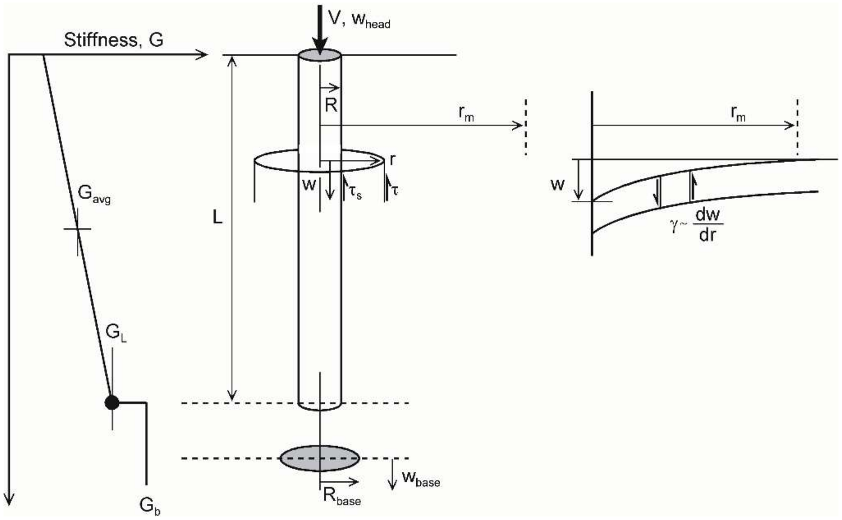
Compatibility:

$$\gamma \approx \frac{dw}{dr}$$

Elasticity:

$$\frac{\tau}{\gamma} = G$$

Integrate to magical radius, r_m, for shaft stiffness, τ_s/w.



Nomenclature for settlement analysis of single piles

Combined response of base (rigid punch) and shaft:

$$\frac{V}{w_{head}} = \frac{Q_b}{w_{base}} + \frac{Q_s}{w} \qquad \frac{V}{w_{head}} = \frac{4R_{base}G_{base}}{1-\nu} + \frac{2\pi L G_{avg}}{\zeta}$$

$$\frac{V}{w_{head}DG_L} = \frac{2}{1-\nu}\frac{G_{base}}{G_L}\frac{D_{base}}{D} + \frac{2\pi}{\zeta}\frac{G_{avg}}{G_L}\frac{L}{D} \qquad \frac{V}{w_{head}DG_L} = \frac{2}{1-\nu}\frac{\eta}{\xi} + \frac{2\pi}{\zeta}\rho\frac{L}{D}$$

These expressions are simplified using dimensionless variables:

Base enlargement ratio, eta $\eta = R_{base}/R = D_{base}/D$ Slenderness ratio L/D

Stiffness gradient ratio, rho $\rho = G_{avg}/G_L$ Base stiffness ratio, xi $\xi = G_L/G_{base}$

It is often assumed that the dimensionless zone of influence, $\zeta = \ln(r_m/R) = 4$.

More precise relationships, checked against numerical analysis are:

$$\zeta = \ln\left\{\{0.5 + (5\rho(1-\nu) - 0.5)\xi\}\frac{L}{D}\right\} \qquad \text{for } \xi=1\text{:} \qquad \zeta = \ln\left\{5\rho(1-\nu)\frac{L}{D}\right\}$$

6.2 Settlement of a compressible pile

$$\frac{V}{w_{head}DG_L} = \frac{\dfrac{2\eta}{(1-\nu)\xi} + \rho\dfrac{2\pi}{\zeta}\dfrac{\tanh\mu L}{\mu L}\dfrac{L}{D}}{1 + \dfrac{1}{\pi\lambda}\dfrac{8\eta}{(1-\nu)\xi}\dfrac{\tanh\mu L}{\mu L}\dfrac{L}{D}} \qquad \text{where } \mu = \frac{\sqrt{8/\zeta\lambda}}{D} \quad \text{Pile compressibility}$$

$\lambda = E_p/G_L$ Pile-soil stiffness ratio

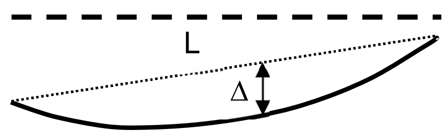
Pile head stiffness, $\frac{V}{w_{head}}$, is maximum when $L \geq 1.5D\sqrt{\lambda}$

Section 7: Damage to buildings from differential settlement

Relative displacement Δ/L:

L is the length of building segment with consistent sagging or hogging

Δ is the maximum settlement of the deformed segment relative to chord L



Distortion and maximum tensile strain ε_{max} in elastic beams of various E/G and L/H:

$\frac{\Delta}{L\,\varepsilon_{max}} \approx$ 1.0 to 1.5 diagonally in end panels due to shear

0.75 to 1.0 longitudinally due to sagging beam

0.25 to 0.5 longitudinally due to hogging beam

Onset of visible (~ 0.1mm) cracks in brick or blockwork walls: $\varepsilon_{max} \approx 0.75\ .10^{-3}$

(Burland & Wroth, 1974)

Categories of associated building damage:

Cat.	Limit	Relative displacement	Description	Action
0	-	$\Delta/L \leq 0.5\ 10^{-3}$	negligible	none
1	SLS	$0.5\ 10^{-3} < \Delta/L \leq 0.75\ 10^{-3}$	very slight	redecorate interior
2	SLS	$0.75\ 10^{-3} < \Delta/L \leq 1.5\ 10^{-3}$	slight	+ some repointing
3	SLS	$1.5\ 10^{-3} < \Delta/L \leq 3.\ 10^{-3}$	moderate	+ significant repointing etc
4	ULS	$3.\ 10^{-3} < \Delta/L \leq 10^{-2}$	severe	shore; consider demolition
5	ULS	$10^{-2} < \Delta/L$	very severe	demolish

(Boscardin & Cording, 1989)

Section 8: Cylindrical cavity expansion

Expansion $\delta A = A - A_o$ caused by increase of pressure $\delta\sigma_c = \sigma_c - \sigma_o$

At radius r: small displacement $\rho = \dfrac{\delta A}{2\pi r}$

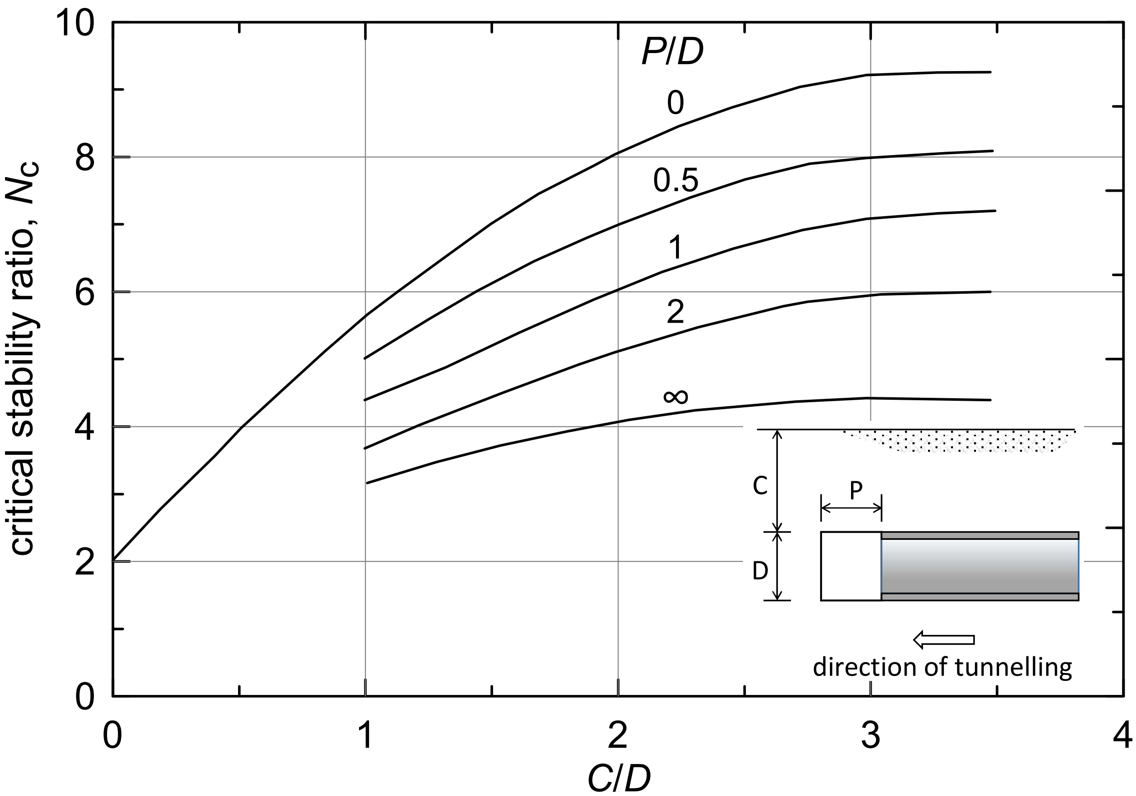
small shear strain $\gamma = \dfrac{2\rho}{r}$

Radial equilibrium: $r\dfrac{d\sigma r}{dr} + \sigma_r - \sigma_\theta = 0$

Elastic expansion (small strains) $\delta\sigma_c = G\dfrac{\delta A}{A}$

zUndrained plastic-elastic expansion $\delta\sigma_c = c_u\left[1 + \ln\dfrac{G}{c_u} + \ln\dfrac{\delta A}{A}\right]$

Section 9: Tunnel Face Stability



Section 10: Ground Movements around tunnels

$$w = w_{max} e^{-\frac{1}{2}\left(\frac{x}{i}\right)^2}$$

$$i = K z_0$$

Where z_0 is the depth of the axis of a tunnel.

K is 0.65 for soft clay, 0.45 for stiff clay and 0.25 for sand above the water table.