4. Shallow Foundation (浅基礎)

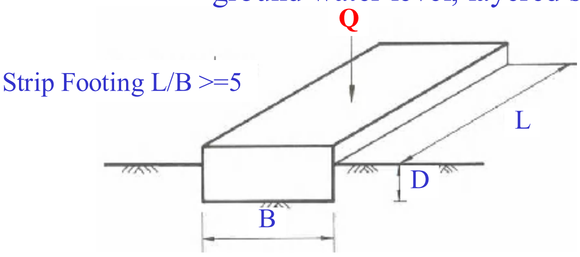
•**Foundation**: rigid footing(剛) ex) mat F., raft F., spread F.
flexible uniform loading(柔), ex) embankment, tank

•**2D and 3D**: 2D strip footing (L>>B)
3D rectangular, circular footing

•**Loading condition**: central vertical loading
eccentric vertical loading(偏心荷重=>モーメント)
inclined loading (傾斜荷重)

•**Embedment(根入)**: D < B

•**Ground condition**: sandy soil (ϕ'), clayey soil(ϕ_u=0), compressibility
ground water level, layered soil,

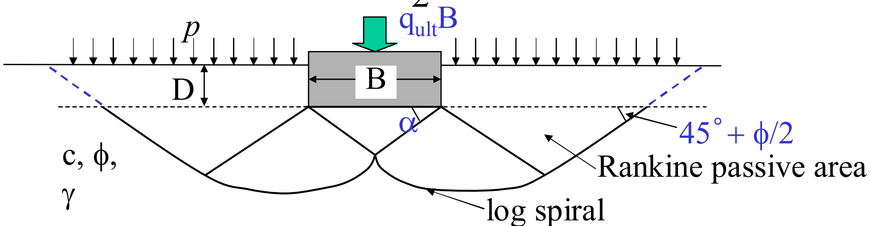


4.1 Bearing capacity

4.1.1Bearing capacity equation

Terzaghi originally introduced bearing capacity equation and gave the **bearing capacity factors** (***N_c,N_q, N_γ***) using limit equilibrium method for 2D under center vertical loading.

$$q_{ult} = Q_{ult}/B = cN_c + q_s N_q + \frac{\gamma B}{2} N_\gamma \qquad q_s = \gamma D + p \qquad (1)$$



Derivation of q_{ult}: superimposing $q_{ult}=q_{ult1} + q_{ult2}$
q_{ult1}: bearing capacity of soil with c, $q_s \neq 0, \gamma=0$
q_{ult2}: bearing capacity of soil with $\gamma \neq 0$, c= q_s =0

Terzaghi bearing capacity factors: $\alpha=\phi$

<u>for rough base</u>:

$$N_q = \frac{1}{1-\sin\phi}\exp\left\{\left(\frac{3}{2}\pi-\phi\right)\tan\phi\right\}$$

$$N_c = (N_q - 1)\cot\phi \qquad (2)$$

$$N_\gamma \approx (N_q - 1)\tan(1.4\phi)$$

<u>for smooth base</u>: (Table3.1, p158, Das text book)

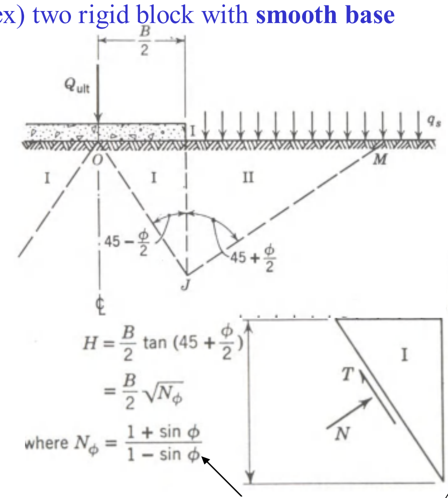
$$N_{q0} = \frac{1+\sin\phi}{1-\sin\phi}\exp(\pi\tan\phi)$$

$$N_{c0} = (N_{q0} - 1)\cot\phi \qquad (3)$$

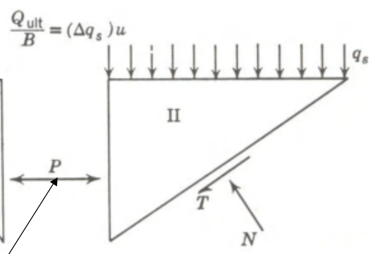
$$N_{\gamma 0} = \frac{1}{2}N_\gamma = \frac{1}{2}(N_{q0} - 1)\tan(1.4\phi)$$

Examples of derivation of bearing capacity factors using limit equilibrium method

ex) two rigid block with **smooth base**



1) Assuming of failure mechanism
2) Calculating forces acting to the rigid body
3) Equilibrium calculations with bearing pressure



K_p Rankine pressure ⇔smooth condition

Equilibrium at the interface between two blocks

Maximum force P that can be applied to passive wedge II

from Rankine's passive pressure equation

$$P = q_s H N_\phi + \frac{1}{2}\gamma H^2 \underline{N_\phi} \quad \leftarrow K_p$$

$$P = q_s \frac{B}{2} N_\phi^{3/2} + \frac{1}{8}\gamma B^2 N_\phi^2 \qquad (4)$$

Maximum average pressure Q_{ult}/B that can be applied to active wedge I

$$P = \frac{Q_{ult}}{B}\frac{H}{N_\phi} + \frac{1}{2}\gamma H^2 \frac{1}{N_\phi}$$

$$\frac{Q_{ult}}{B} = \frac{P}{H} N_\phi - \frac{1}{2}\gamma H = \left(\frac{2P}{B} - \frac{1}{4}\gamma B\right)\sqrt{N_\phi} \qquad (5)$$

(4)=>(5)

$$\frac{Q_{ult}}{B} = q_s N_\phi^2 + \frac{1}{4}\gamma B N_\phi^{5/2} - \frac{1}{4}\gamma B N_\phi^{1/2}$$

$$\frac{Q_{ult}}{B} = q_s N_\phi^2 + \frac{\gamma B}{4}\left(N_\phi^{5/2} - N_\phi^{1/2}\right) \qquad (6)$$

$$\left.\begin{aligned} N_q &= N_\phi^2 = K_p^2 \\ N_\gamma &= \left(N_\phi^{5/2} - N_\phi^{1/2}\right)/2 \end{aligned}\right\} (7)$$

$$N_\phi = K_p = \frac{1+\sin\varphi}{1-\sin\phi} = \tan^2\left(45^\circ + \frac{\phi}{2}\right) \qquad (8)$$

N_q and N_γ in eq.(7) are smaller than value derived by other method.
compare eqs. (3) and (7)

ex) for $\phi=30^\circ$ $N_{q(3)}=18$, $N_{q(7)}=9$;
for $\phi=40^\circ$ $N_{q(3)}=64$, $N_{q(7)}=21$

Why??

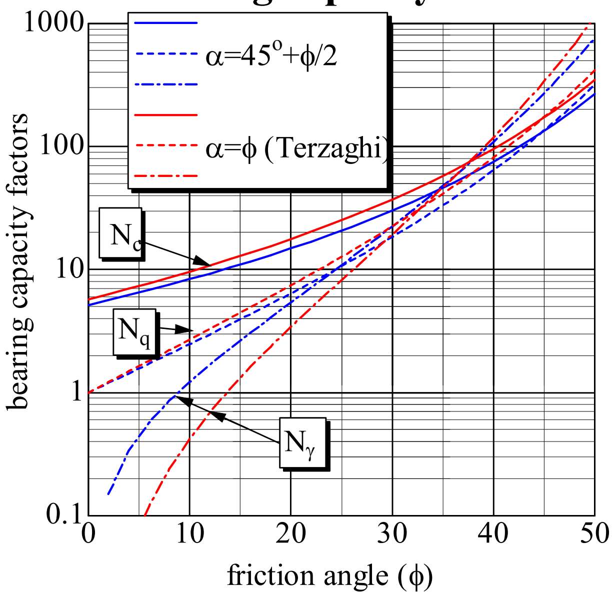
Bearing capacity factors: α=45°+ϕ/2

for rough base:

$$\left.\begin{aligned} N_q &= \tan^2\left(45^\circ + \frac{\phi}{2}\right)\exp(\pi \tan\phi) \\ N_c &= (N_q - 1)\cot\phi \\ N_\gamma &\approx 2(N_q + 1)\tan\phi \end{aligned}\right\} \quad (9)$$

N_q, N_c, N_γ are given by tabulated form or chart.
(e.g., Das "Principle of Foundation Eng.")

Bearing capacity factors



4.1.2 Effect of footing shape, eccentricity, inclined load, embedment

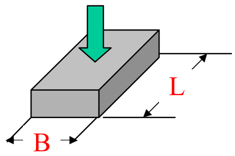
General bearing capacity equation

$$q_{ult} = cN_c F_{cs} F_{cd} F_{ci} + q_s N_q F_{qs} F_{qd} F_{qi} + \frac{\gamma B}{2} N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \quad (10)$$

F_{cs}, F_{qs}, $F_{\gamma s}$: shape factors（形状係数） ***3D effects***

F_{cd}, F_{qd}, $F_{\gamma d}$: depth factors（深さ係数）

F_{ci}, F_{qi}, $F_{\gamma i}$: inclination factors （傾斜荷重係数）



Shape factors: ***semi-empirical***

Meyerhof(1963)

$$F_{cs} = 1 + 0.2 N_\phi \frac{B}{L}$$ 3D>2D

$$F_{qs} = 1.0 \qquad (\phi = 0^\circ)$$

$$F_{qs} = F_{\gamma s} = 1 + 0.1 N_\phi \frac{B}{L} \qquad (\phi > 10^\circ)$$ (11)

3D>2D

$$N_\phi = \tan^2\left(\frac{1}{4}\pi + \frac{1}{2}\phi\right)$$

Hansen(1970)

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c}$$

$$F_{qs} = 1 + \frac{B}{L}\tan\phi$$ (12)

$$F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

3D<2D

for circular footing: B/L=1

Theoretical values of shape factors

Bearing capacity of circular footing (**B/L~1**)can be solved by slip line method using cylindrical coordinate.

About F_{cs}

q_{ult} of circular footing on ϕ_u=0 material: 6.05c_u => F_{cs}=6.06/5.14=1.18

good agreement

using eqs.(11) and (12) with ϕ_u=0 and B/L=1, F_{cs}=1.2 and 1.19

About $F_{\gamma s}$,

Slip line method gives larger N_γ for circular F. than strip F. for the same ϕ value. That means $F_{\gamma s}$ >1, which is consistent with eq.(11) and inconsistent with eq.(12).

Key words to explain N_γ: stress dependency of ϕ', strain constraint (or σ'_2) effect on ϕ' and progressive failure or local failure.

Slip Line Method for axisymmetric condition

two stress equilibrium equations with cylindrical coordinate

$$\frac{\partial \sigma_r}{\partial r} + \frac{\partial \tau_{rz}}{\partial z} + \frac{\sigma_r - \sigma_\theta}{r} = 0$$

$$\frac{\partial \tau_{rz}}{\partial r} + \frac{\partial \sigma_z}{\partial z} + \frac{\tau_{rz}}{r} = \gamma$$

○ **four unknown values**

three equations

failure criteria (Mohr-Coulomb)

$\sigma_1 - \sigma_3 = c_u$ for undrained conditions (ϕ_u=0)

$\sigma'_1 - \sigma'_3 = 2c' \cos\phi' + (\sigma'_1 + \sigma'_3)\sin\phi'$ for drained conditions

need one more assumption: $\sigma_\theta = \sigma_1$ or σ_3 : Haar & Karman's assumption

As a common assumption, $\sigma_\theta = \sigma_3$ is the most reliable

geometric condition

$$+ \quad \alpha(s_1)line: \frac{dr}{dz} = \tan(\eta - 45^\circ + \frac{\phi'}{2})$$

$$\beta(s_2)line: \frac{dr}{dz} = \tan(\eta + 45^\circ - \frac{\phi'}{2})$$

⇒ two differential eqs. expressing ***s*** and η along each slip line in terms of ***s***, rotation of η and position of (***r***,***z***).

N_γ obtained from slip line methods

Bolton et al. (1991), Can. Geotech. Vol.30, p.1024-1033.

ϕ	$\tan\phi$	N_γ smooth base strip	N_γ smooth base circular	N_γ rough base strip	N_γ rough base circular
5	0.09	0.09	0.06	0.62	0.68
10	0.18	0.29	0.21	1.71	1.37
15	0.27	0.71	0.60	3.17	2.83
20	0.36	1.60	1.30	5.97	6.04
25	0.47	3.51	3.00	11.6	13.5
30	0.58	7.74	7.10	23.6	31.9
31	0.60	9.1	8.6	27.4	38.3
32	0.62	10.7	10.3	31.8	46.1
33	0.65	12.7	12.4	37.1	55.7
34	0.67	15.0	15.2	43.5	67.6
35	0.70	17.8	18.2	51.0	82.4
36	0.73	21	22	60	101
37	0.75	25	27	71	124
38	0.78	30	33	85	153
39	0.81	36	40	101	190
40	0.84	44	51	121	238
41	0.87	53	62	145	299
42	0.90	65	78	176	379
43	0.93	79	99	214	480
44	0.97	97	125	262	619
45	1.00	120	160	324	803
46	1.04	150	210	402	1052
47	1.07	188	272	505	1384
48	1.11	237	353	638	1847
49	1.15	302	476	815	2491
50	1.19	389	621	1052	3403

$N_{\gamma c}/N_{\gamma p} = F_{\gamma s}$

0.86 (31.9 / 37.1)

0.84 (101 / 121)

$F_{\gamma s}$<1 consistent with eq.(12)

for same ϕ value

N_γ of strip (2D)

^

N_γ of circular (3D)

⇓

$F_{\gamma s} > 1$

consistent with eq.(11)

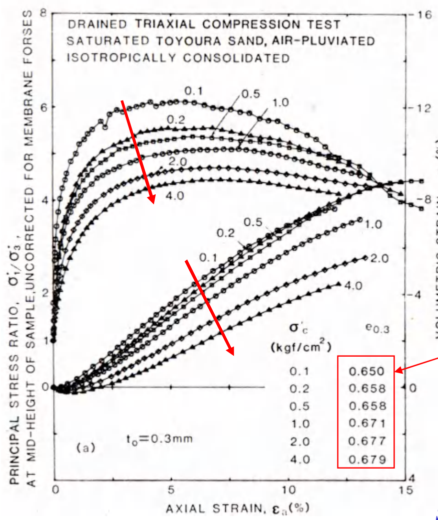
inconsistent with eq.(12)

$\phi'_{plane} \sim 1.1\ \phi'_{triaxial}$

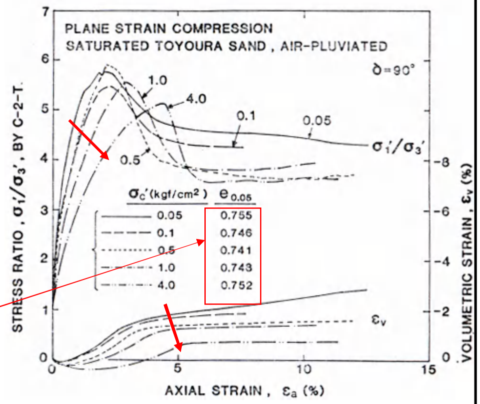
at small pressure: difference of ϕ is more: => smaller $F_{\gamma s}$ from ***small scale test***

Stress dependency of stress-strain relations

Tatsuoka et al., S&F, Vo.26, No.1, p.65-84, 1984



↓ σ'3, σ'c increasing

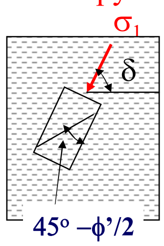


The larger confining stress, the smaller stress ratio and less dilation.

Relation ship between ϕ' and σ'_3

Tatsuoka et al., S&F, Vo.26, No.1, p.65-84, 1984

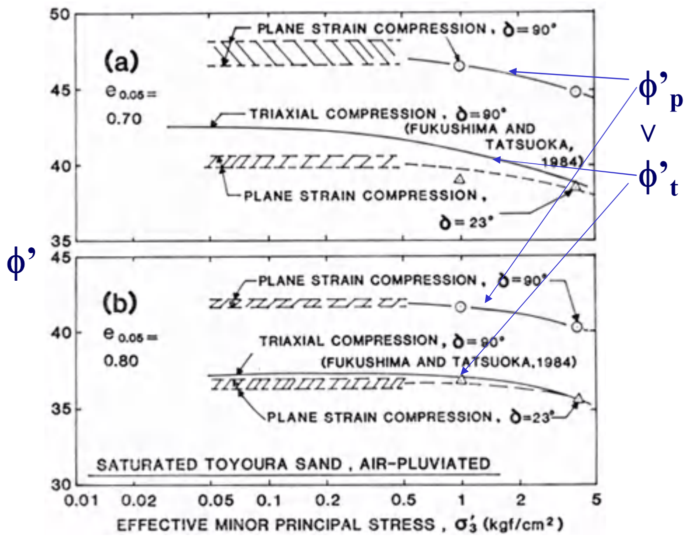
anisotropy of ϕ'



Strain constraint effect leads

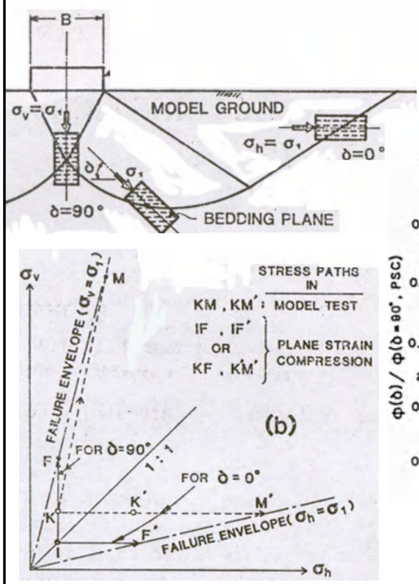
$\sigma_2 > \sigma_3$ in plane strain;

$\sigma_2 = \sigma_3$ in triaxial comp.



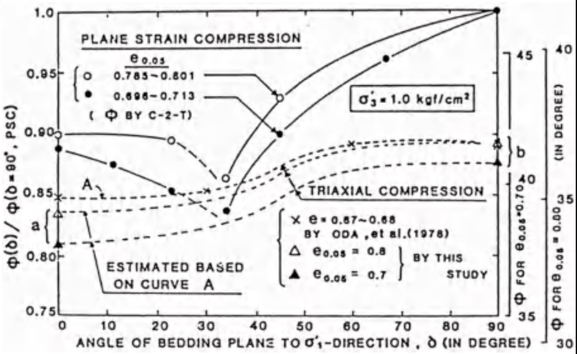
stress dependency of ϕ'

Anisotropy of ϕ'



The denser, the higher anisotropy and the larger difference betw. P and T.

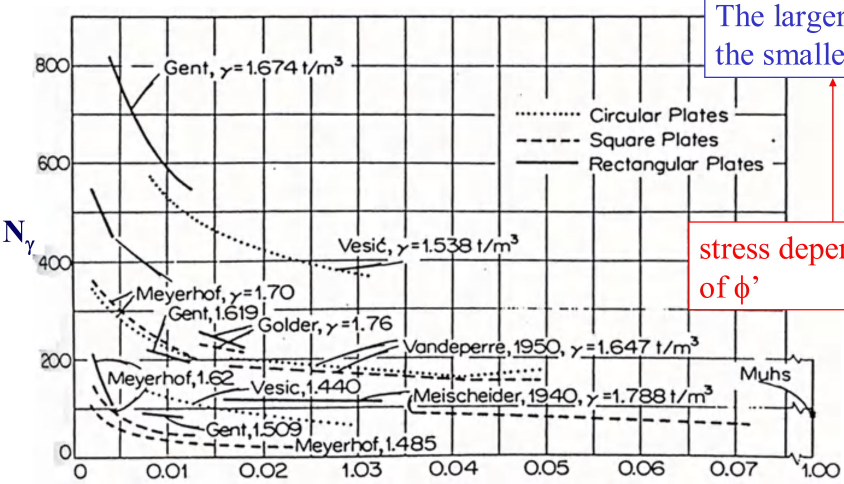
PS is higher anisotropy than TA.



Tatsuoka et al., S&F, Vo.26, No.1, p.65-84, 1984

Scale effects of N_γ

–Effect of size of surface footing in sand-



The larger, the smaller N_γ

stress dependency of ϕ'

γB (kgf/cm²) De Beer (1965)

Interpretation of the difference between eq.(11) and (12)

$F_{\gamma s}$ in eq. (12) was obtained from the model test, where mobilized ϕ' value is higher for the sand under strip footings than circular footings.

$F_{\gamma s}$ in eq. (11) can be applied in the assumption that the mobilized ϕ' values are the same for strip footings and circular footings.

"Which equation is better?? " depends on the ϕ' value used in the analysis, ϕ'_p or ϕ'_t?. If used ϕ' is ϕ'_p, eq.(12) is better and if ϕ'_t, eq.(11) might be better.

As in actual design practice, the ϕ' values are often estimated from empirical formulation (e.g., using N value) and not so accurate.
For safety reason, eq.(12) is normally used as the scale factor of N_γ.

Depth factors

Hansen(1970)

D/B < 1

$$\left.\begin{aligned} F_{cd} &= 1 + 0.4\frac{D}{B} \\ F_{qd} &= 1 + 2\tan\phi(1-\sin\phi)^2\frac{D}{B} \\ F_{\gamma d} &= 1 \end{aligned}\right\}(13)$$

D/B > 1

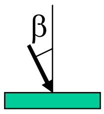
$$\left.\begin{aligned} F_{cd} &= 1 + 0.4\tan^{-1}\left(\frac{D}{B}\right) \\ F_{qd} &= 1 + 2\tan\phi(1-\sin\phi)^2\tan^{-1}\left(\frac{D}{B}\right) \\ F_{\gamma d} &= 1 \end{aligned}\right\}(14)$$

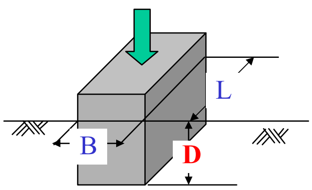
in radius

Inclination factors

Meyerhof(1963)

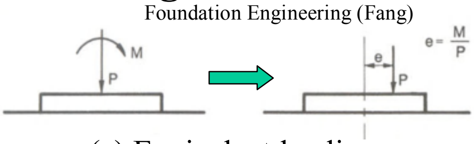
$$\left.\begin{aligned} F_{ci} &= F_{qi} = \left(1-\frac{\beta^\circ}{90^\circ}\right)^2 \\ F_{\gamma i} &= \left(1-\frac{\beta}{\phi}\right)^2 \end{aligned}\right\}(15)$$



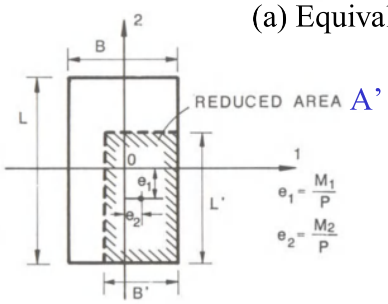


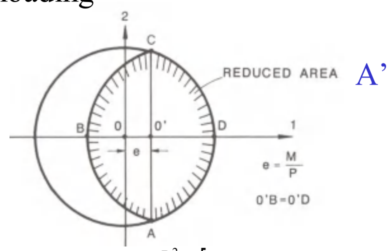
Reduced footing area for eccentric load

Foundation Engineering (Fang)



(a) Equivalent loading





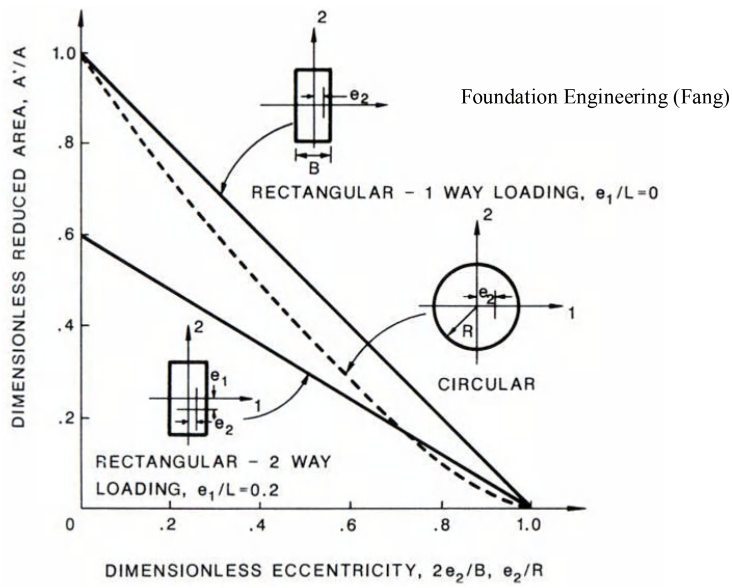
$$L \Rightarrow L' = L - 2e_1, \quad B \Rightarrow B' = B - 2e_2 \quad (16)$$

(b) Rectangular footing

$$A' = 2S = B'L', \quad S = \frac{\pi R^2}{2} - \left[e\sqrt{R^2 - e^2} + R^2 \arcsin(e/R) \right]$$

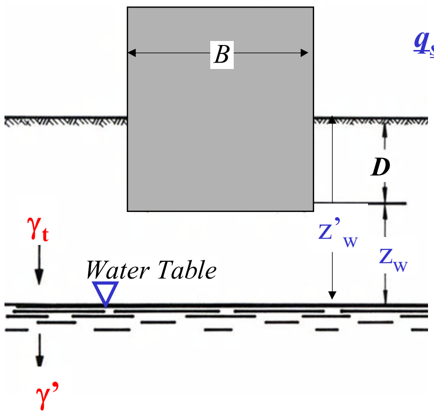
$$L' = \left[2S \left(\frac{R+e}{R-e} \right)^{0.5} \right]^{0.5}, \quad B' = L' \left(\frac{R+e}{R-e} \right)^{0.5} \quad (17)$$

(c) Circular footing



Area reduction factors for eccentrically loaded footings

4.1.3 Effect of water table in sandy soil
- effective stress -



$$q_{ult} = Q_{ult} / B = cN_c + q_s N_q + \frac{\gamma B}{2} N_\gamma$$

q_s in the second term of the BC equation

$$z'_w < D : q_s \Rightarrow z'_w \gamma_t + (D - z'_w)\gamma'$$
$$D < z_w : q_s \Rightarrow D\gamma_t \qquad (18)$$

γ in the last term of the BC equation

$$z_w < 0 : \gamma \Rightarrow \gamma'$$
$$0 \le z_w \le B : \gamma \Rightarrow \gamma' + \frac{z_w}{B}(\gamma_t - \gamma')$$
$$B \le z_w : \gamma \Rightarrow \gamma_t \qquad (19)$$

4.1.4 Effect of compressibility

dense => general failure

loose => local failure , punching shear failure ➡ *settlement is more important than q_{ult} for loose soils.*

φ'>ψ: The looser, the smaller q_{ult}.

=> reduction of φ' value

φ in the BC eq. $\phi'_m \Rightarrow \tan^{-1}\left(\frac{2}{3}\tan\phi'\right)$ (20) by Terzaghi

bearing capacity factors for loose soils: N'_c, N'_q, N'_γ

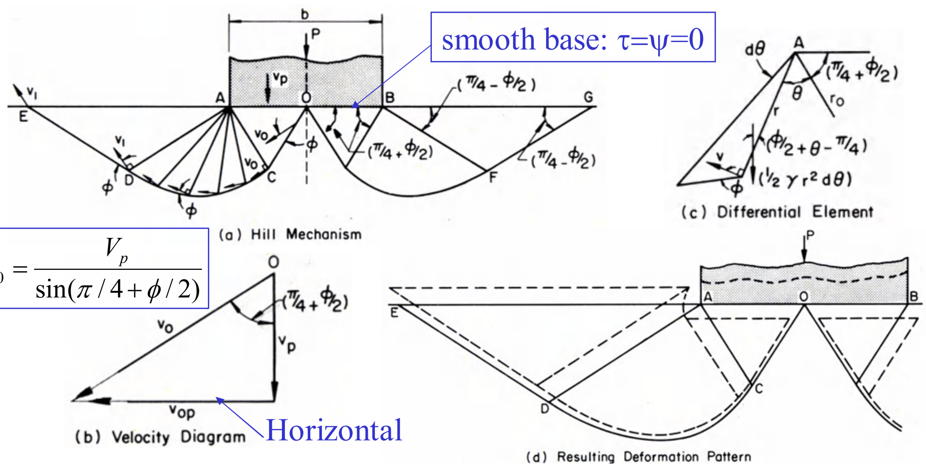
(Table3.2, p160, Das text book)

Modified f value by Davis (1968)

$$\tan\phi'_m = \frac{\cos\psi \sin\phi'}{1 - \sin\psi \sin\phi'} \qquad (21)$$

4.1.4 Effect of footing base roughness

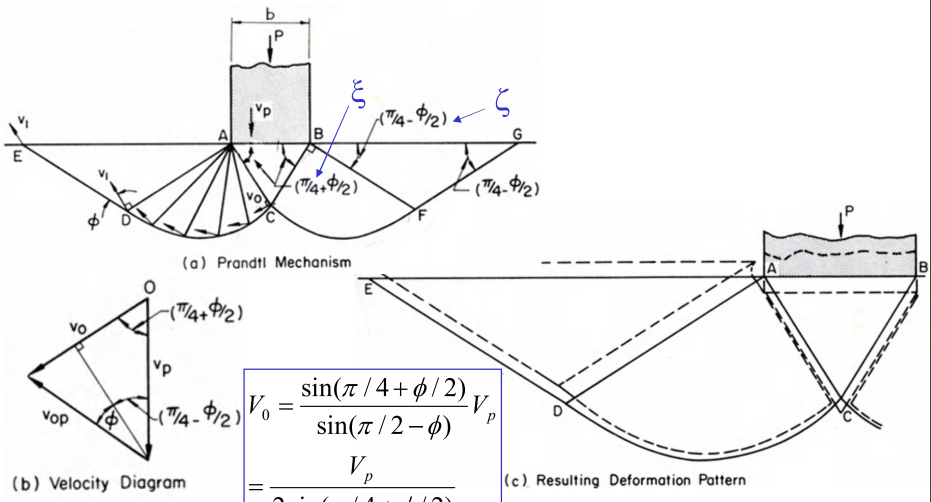
Hill Mechanism used for bearing capacity calculation of rigid footing with **smooth** base “Limit analysis and soil plasticity” Chen, (1975)



$$V_0 = \frac{V_p}{\sin(\pi/4+\phi/2)}$$

(a) Hill Mechanism (b) Velocity Diagram (c) Differential Element (d) Resulting Deformation Pattern

Prandtl Mechanism used for bearing capacity calculation of rigid footing with **rough** base “Limit analysis and soil plasticity” Chen, (1975)



$$V_0 = \frac{\sin(\pi/4+\phi/2)}{\sin(\pi/2-\phi)}V_p = \frac{V_p}{2\sin(\pi/4+\phi/2)}$$

(a) Prandtl Mechanism (b) Velocity Diagram (c) Resulting Deformation Pattern

•N_γ for smooth footing obtained from Hill Mechanism

$$[N_\gamma]_{Hill} = \frac{1}{4}\tan\left(\frac{1}{4}\pi + \frac{1}{2}\phi'\right)\left[\tan\left(\frac{1}{4}\pi + \frac{1}{2}\phi'\right)\exp\left(\frac{3}{2}\pi\tan\phi'\right) - 1\right] \quad (22)$$

$$+\frac{3\sin\phi'}{1+8\sin^2\phi'}\left\{\left[\tan\left(\frac{1}{4}\pi + \frac{1}{2}\phi'\right) - \frac{\cot\phi'}{3}\right]\exp\left(\frac{3}{2}\pi\tan\phi'\right) + \tan\left(\frac{1}{4}\pi + \frac{1}{2}\phi'\right)\frac{\cot\phi'}{3} + 1\right\}$$

$[N_\gamma]_{Hill}$: 0.72 (φ'=10°) ,3.45 (φ'=20°), 15.2 (φ'=30°) ,81.8 (φ'=40°)

•N_γ for rough footing obtained from Prandtl Mechanism

$$[N_\gamma]_{Prandtl} = 2[N_\gamma]_{Hill} \quad \text{for } \xi = 45° + \phi'/2 \text{ and } \zeta = 45° - \phi'/2 \quad (23)$$

The least $[N_\gamma]_{Prandtl}$ is given by $\frac{\partial [N_\gamma]_{Prandtl}}{\partial \xi} = 0, \frac{\partial [N_\gamma]_{Prandtl}}{\partial \zeta} = 0$

ex) $[N_\gamma]_{Prandtl}$ =26.6 for φ'=30° when ξ = 46° and ζ = 45°-φ'/2=30°

4.2Settlement of Foundation

4.2.1 Stress due to loading

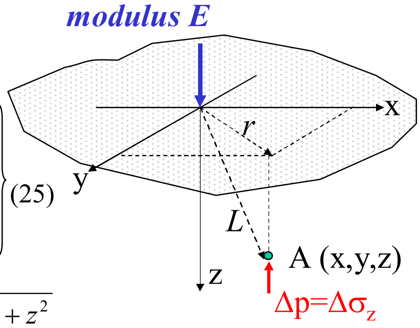
Stresses due to a concentrated load (Boussinesq equation) on liner elastic halfspace

vertical stress is independent on Poisson's ratio ν and Young's modulus E

$$\Delta p = \Delta\sigma_z = \frac{3Pz^3}{2\pi L^5} = \frac{3P}{2\pi}\frac{z^3}{\left[r^2+z^2\right]^{5/2}} \quad (24)$$

$$\left.\begin{aligned}
\Delta\sigma_x &= \frac{P}{2\pi}\left\{\frac{3x^2z}{L^5} - (1-2\nu)\left[\frac{x^2-y^2}{Lr^2(L+z)} + \frac{y^2z}{L^3r^2}\right]\right\} \\
\Delta\sigma_y &= \frac{P}{2\pi}\left\{\frac{3x^2z}{L^5} - (1-2\nu)\left[\frac{y^2-x^2}{Lr^2(L+z)} + \frac{x^2z}{L^3r^2}\right]\right\} \\
\Delta\tau_{rz} &- \frac{3Qrz^2}{2\pi r^5}
\end{aligned}\right\} (25)$$

$$r = \sqrt{x^2+y^2} \qquad L = \sqrt{x^2+y^2+z^2} = \sqrt{r^2+z^2}$$



Vertical stress due to surface loading with uniform pressure Δq_0

integration of eq.(24)

circular loading (center)

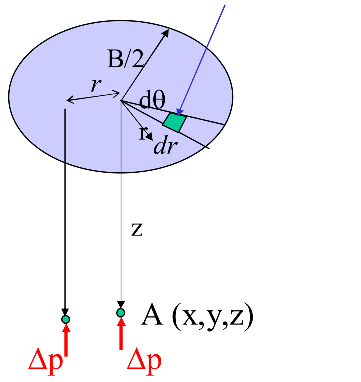
point load acting on *small area*

$$dp = \frac{3(\Delta q_0 r d\theta dr)}{2\pi z^2\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}} \quad (25)$$

$$\Delta p = \int dp = \int_{\theta=0}^{\theta=2\pi}\int_{r=0}^{r=B/2} \frac{3(\Delta q_0 r d\theta dr)}{2\pi z^2\left[1+\left(\frac{r}{z}\right)^2\right]^{5/2}}$$

$$= \Delta q_0\left\{1-\frac{1}{\left[1+\left(B/2z\right)^2\right]^{3/2}}\right\} \quad (26)$$

influential factor: I



(Table4.1, p222, Das text book)
as function of *z/(B/2)* and *r/ (B/2)*

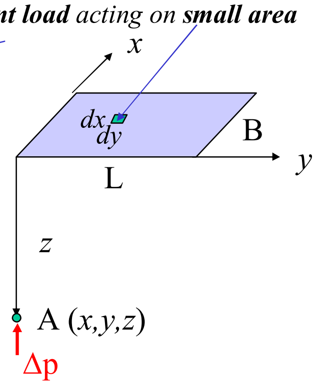
Rectangular loading (at corner)

point load acting on *small area*

$$dp = \frac{3\Delta q_0 (dxdy) z^3}{2\pi\left[x^2+y^2+z^2\right]^{5/2}} \quad (27)$$

$$\Delta p = \int dp = \int_{y=0}^{L}\int_{x=0}^{B} \frac{3\Delta q_0 (dxdy) z^3}{2\pi\left[x^2+y^2+z^2\right]^{5/2}}$$

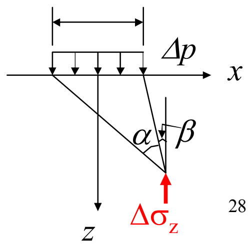
$$= \Delta q_0 I \quad (28)$$



influence factor I=f(m=B/z,n=L/z) ← (Table4.2, p224,225, Figure4.4,p226, Das text book)

Strip loading footing

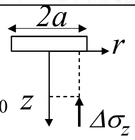
$$\Delta\sigma_z = \frac{\Delta q_0}{\pi}\{\alpha + \sin\alpha(\alpha + 2\beta)\} \quad (29)$$



Influence factors
flexible circular and rectangular footing (at corner)

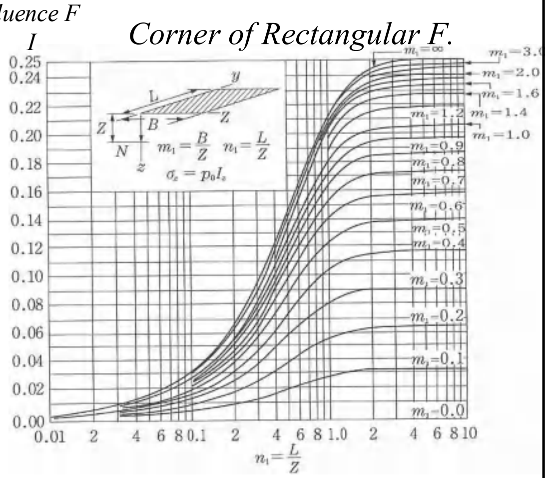
Circular F.

z/a	r/a = 0	0.2	0.4	0.6	0.8	1.0
0	1.000	1.000	1.000	1.000	1.000	1.000
0.1	0.999	0.999	0.998	0.996	0.976	0.484
0.2	0.992	0.991	0.987	0.970	0.890	0.468
0.3	0.976	0.973	0.963	0.922	0.793	0.451
0.4	0.949	0.943	0.920	0.860	0.712	0.435
0.5	0.911	0.902	0.869	0.796	0.646	0.417
0.6	0.864	0.852	0.814	0.732	0.591	0.400
0.7	0.811	0.798	0.756	0.674	0.545	0.367
0.8	0.756	0.743	0.699	0.619	0.504	0.366
0.9	0.701	0.688	0.644	0.570	0.467	0.348
1.0	0.646	0.633	0.591	0.525	0.434	0.332
1.2	0.546	0.535	0.501	0.447	0.377	0.300
1.5	0.424	0.416	0.392	0.355	0.308	0.256
2.0	0.286	0.286	0.268	0.248	0.224	0.196
2.5	0.200	0.197	0.191	0.180	0.167	0.151
3.0	0.146	0.145	0.141	0.135	0.127	0.118
4.0	0.087	0.086	0.085	0.082	0.080	0.075



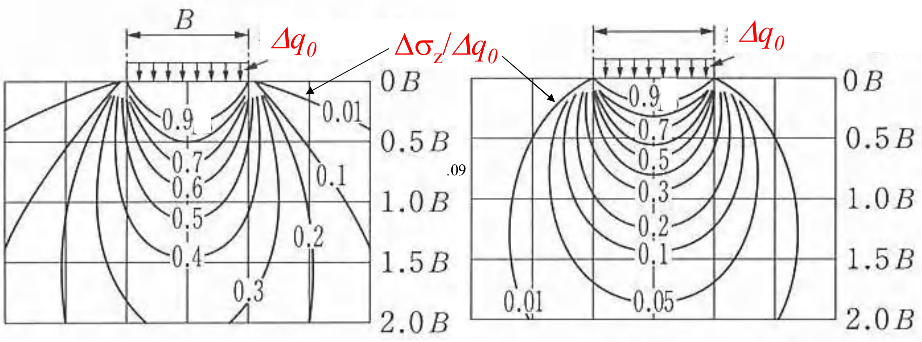
Influence F

Corner of Rectangular F.



Kusakabe"Soil Mechanics" (2004)

Contour of vertical stress σ_z in semi-infinite elastic body subjected to uniform surface loading
ー Stress bulb (応力球根) ー



Strip: 2D

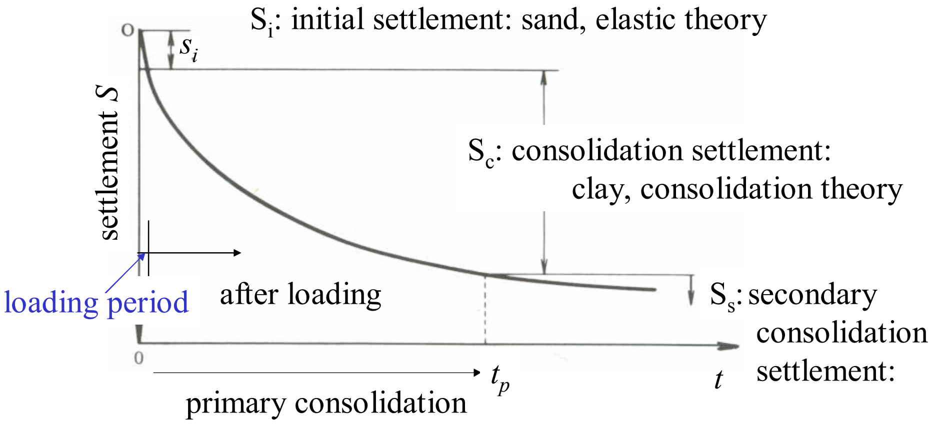
Kusakabe"Soil Mechanics" (2004)

Circular:3D

Deeper propagation

Shallower propagation
~0 at z=2B

4.2.2 Settlement calculation



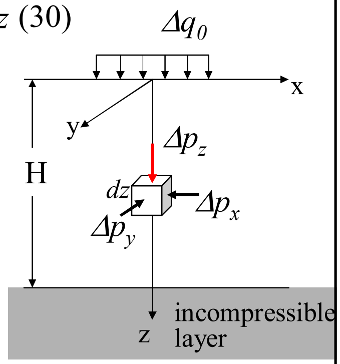
Elastic settlement by elasticity theory

for isotropic elastic media, by Hooke's law

$$S_e = \int_{z=0}^{H} \varepsilon_z dz = \frac{1}{E_s}\int_0^H \left(\Delta p_z - \nu_s \Delta p_x - \nu_s \Delta p_y\right) dz \quad (30)$$

$$S_e = \Delta q_0 B \frac{1-\nu_s^2}{E_s} I_\rho \quad (31)$$

Δq_0: net pressure applied from foundation
B: width of the footing
ν_s: Poisson's ratio of soil
E_s: Young's modulus of soil
I_ρ: non-dimensional influence factor



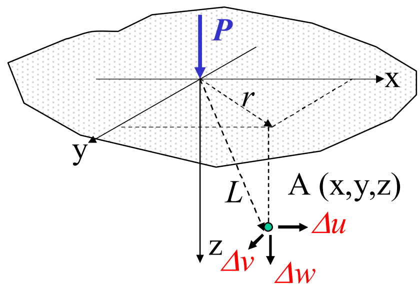
rigid footing =>uniform settlement
flexible footing (uniform loading)
=> non-uniform settlement

Settlement caused by surface load

Boussinesq's equation :

Displacements due to a concentrated load (P) on liner elastic halfspace (*u*,*v*,*w*)

$$\left.\begin{aligned}\Delta w &= \frac{P(1+\nu)}{2\pi E}\left\{\frac{z^2}{L^3}+\frac{2(1-\nu)}{L}\right\}\\ \Delta u &= \frac{P(1+\nu)}{2\pi E}\left\{\frac{z}{L^3}-\frac{(1-\nu)}{L(L+z)}\right\}x\\ \Delta v &= \frac{P(1+\nu)}{2\pi E}\left\{\frac{z}{L^3}-\frac{(1-\nu)}{L(L+z)}\right\}y\end{aligned}\right\} \quad (32)$$



$$r=\sqrt{x^2+y^2} \quad L=\sqrt{x^2+y^2+z^2}=\sqrt{r^2+z^2}$$

Important => vertical displacement (*w*) at surface (z=0,L=r)

$$\Delta w=\frac{P(1-\nu^2)}{\pi E r} \quad (33)$$

Influence factors

uniform

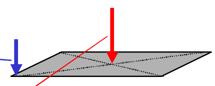
Schleicher's solution (1926) on the influence factor

for the corner of a flexible rectangular footing on infinite half space: (H=∞)

$$\alpha=\frac{1}{\pi}\left[\ln\left(\frac{\sqrt{m^2+1}+m}{\sqrt{m^2+1}-m}\right)+m\ln\left(\frac{\sqrt{m^2+1}+1}{\sqrt{m^2+1}-1}\right)\right] \quad \textit{where } m=L/B \quad (34)$$

$$S_e=\Delta q_0 B\frac{1-\nu_s^2}{E_s}\frac{\alpha}{2} \quad (35)$$

I_ρ



for the center of a flexible rectangular footing

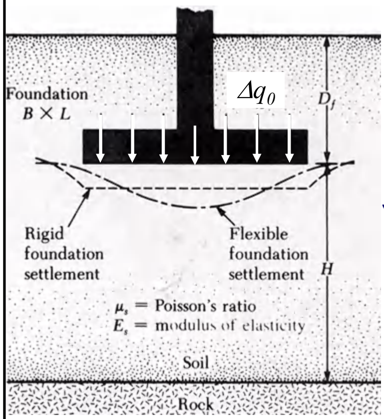
twice??

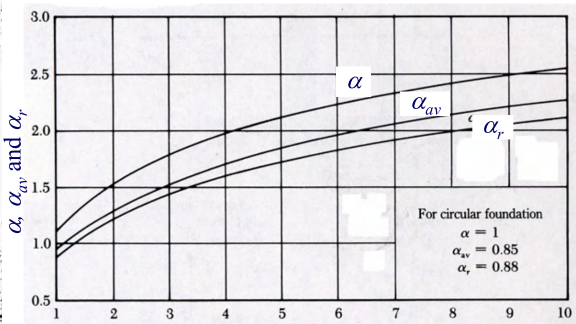
$$S_e=\Delta q_0 B\frac{1-\nu_s^2}{E_s}\alpha \quad (36)$$

settlement of rigid rectangular footing

$$S_e=\Delta q_0 B\frac{1-\nu_s^2}{E_s}\alpha_r \quad (37)$$

Elastic settlement of flexible and rigid foundations

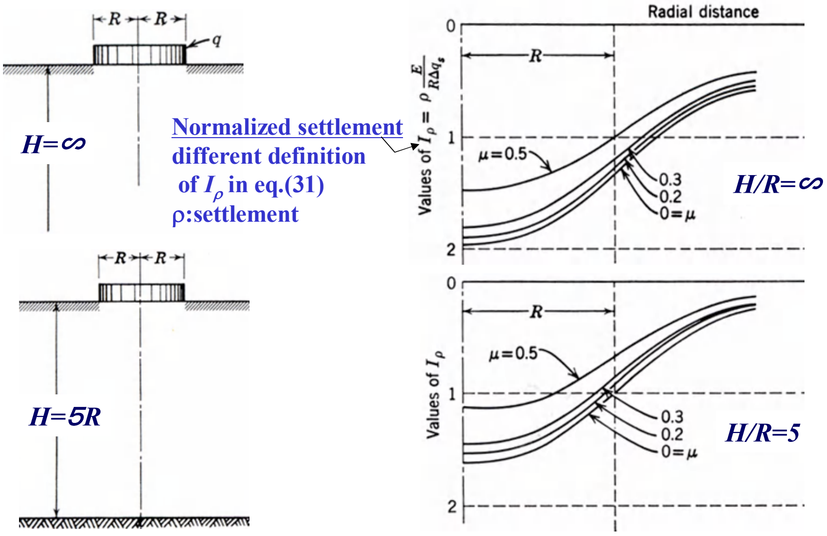




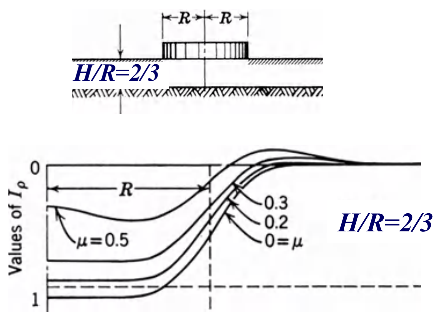
"Principle of Foundation Engineering" B. M. Das (1999)

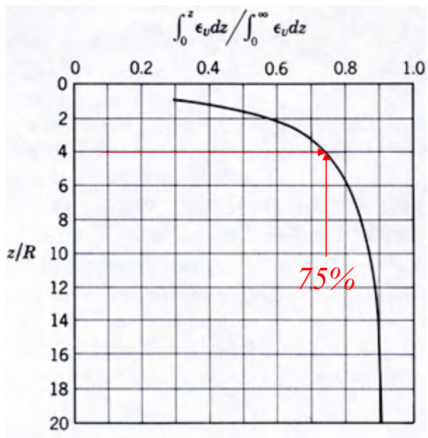
Effect of limited thickness of compressible layer
-Influence factors under uniform circular load-

" Soil Mechanics" Lambe and Whitman



Effect of limited thickness of compressible layer





" Soil Mechanics" Lambe and Whitman

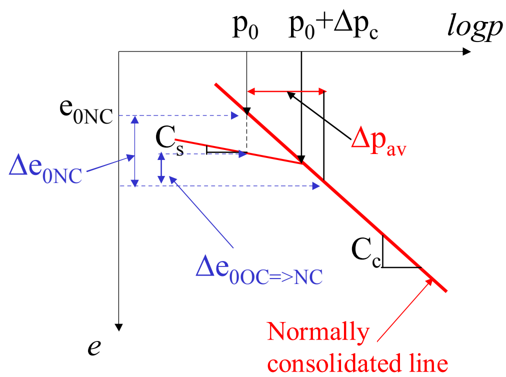
Settlement due to consolidation

$$S_c = \int \varepsilon_z dz \qquad (38)$$

for 1D condition

$$\varepsilon_z = \frac{\Delta e}{1+e_0} \qquad (39)$$

Δp_{av}:
average increase of pressure on the clay layer caused by loading from structure



$$\Delta p_{av} \approx \frac{1}{6}\left(\Delta p_{top} + 4\Delta p_{middle} + \Delta p_{bottom}\right) \qquad (40)$$

Consolidation settlement calculation

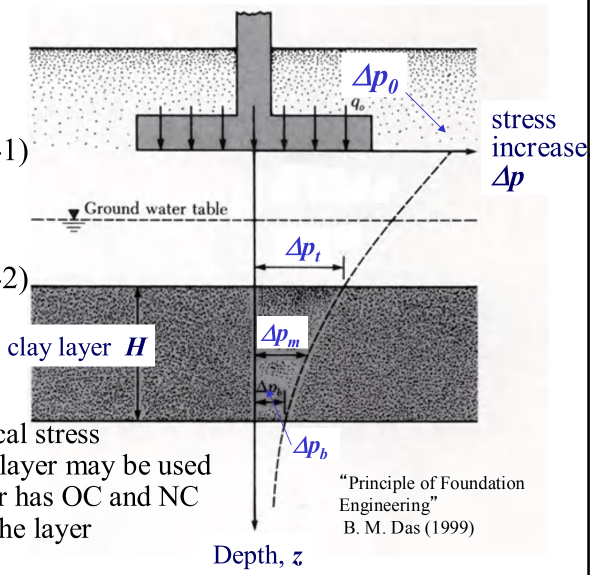
ε_z may be calculated from

$$\varepsilon_z = \frac{C_c}{1+e_0}\log(\frac{p_0+\Delta p_{av}}{p_0}) \quad (41)$$

for NC

$$\varepsilon_z = \frac{C_r}{1+e_0}\log(\frac{p_0+\Delta p_{av}}{p_0}) \quad (42)$$

for OC



p_0:
average initial effective vertical stress
Stress at mid-depth of clay layer may be used as p_0. If clay layer is thick or has OC and NC portion, it is better to divide the layer into some sub-layers.

Field loading test

Estimation of settlement of Footing with B=B_F
using load settlement relation from Plate Load Test with B=B_P

settlement of footing (B=B_F)in a design

settlement in plate load test (B=B_P)

for clayey soil:

eq.(1)

$$q_{ult(F)} = q_{ult(P)} \Rightarrow S_F = S_P\frac{B_F}{B_P} \quad (43)$$

E:constant

for sandy soil:

$$q_{ult(F)} = q_{ult(P)}\frac{B_F}{B_P} \Rightarrow S_F = S_P\left(\frac{2B_F}{B_F+B_P}\right)^2 \quad (44)$$

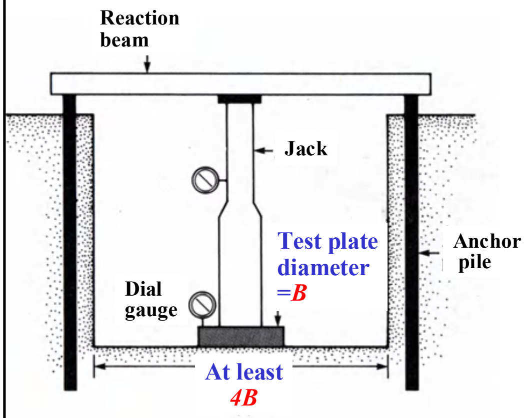
empirical

E: increasing with depth but not linearly

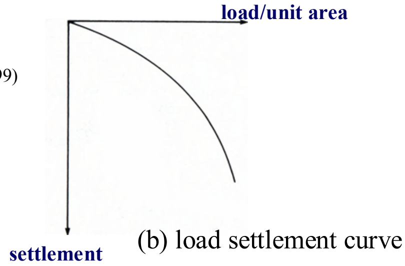
Terzaghi and Peck(1967)

Plate load test

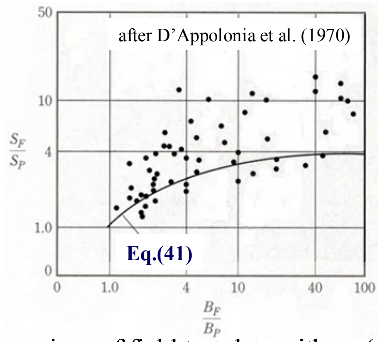
"Principle of Foundation Engineering" B. M. Das (1999)



(a)test arrangement



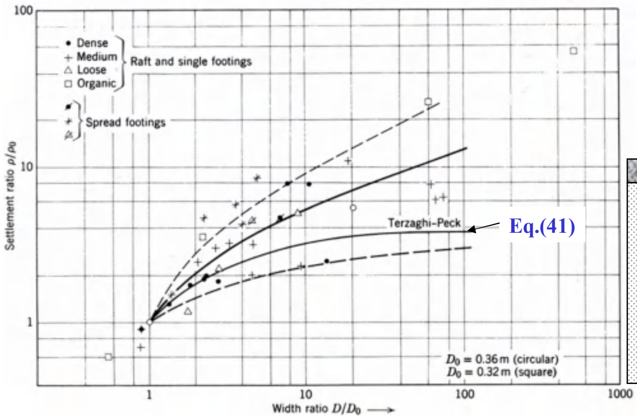
(b) load settlement curve

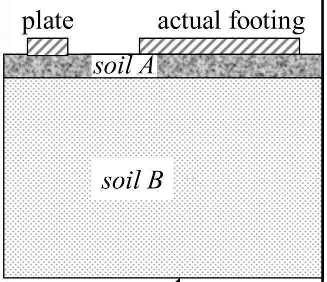


(c) comparison of field test data with eq.(44)

Comparison between settlement and dimension ratio of plate and footing

" Soil Mechanics" Lambe and Whitman



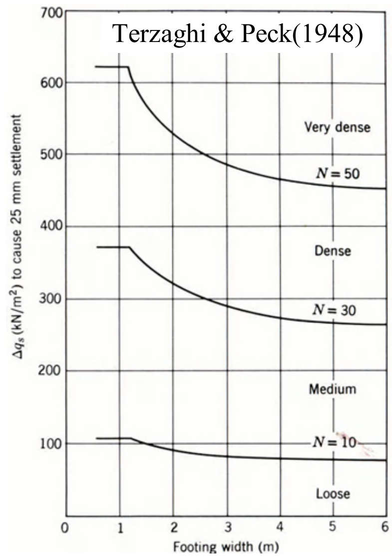


Situation where plate load test results can be misleading

Prediction of settlement by penetration test

" Soil Mechanics" Lambe and Whitman

Terzaghi & Peck(1948)



Settlement of footing from standard penetration test N

Meyerhof (1965)

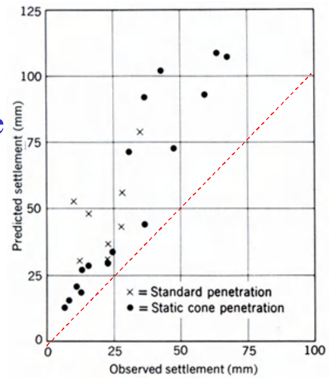
$$\Delta q_s = 0.47 N S_F \qquad B \le 1.2m$$

$$\Delta q_s = 0.31 N S_F \left(\frac{B+0.3}{B}\right)^2 \qquad B > 1.2m$$

kN/m^2 mm m

over estimate

conservative (safety side)

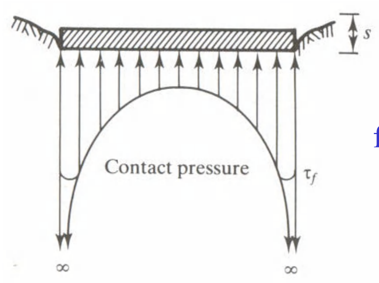


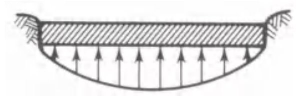
Distribution of settlement and contact stress for rigid and flexible footing on cohesive (ϕ=0) and cohesionless (ϕ>0) material

" Soil Mechanics" Lambe and Whitman

cohesive (ϕ=0) material

granular (ϕ>0) material





rigid footing

flexible footing