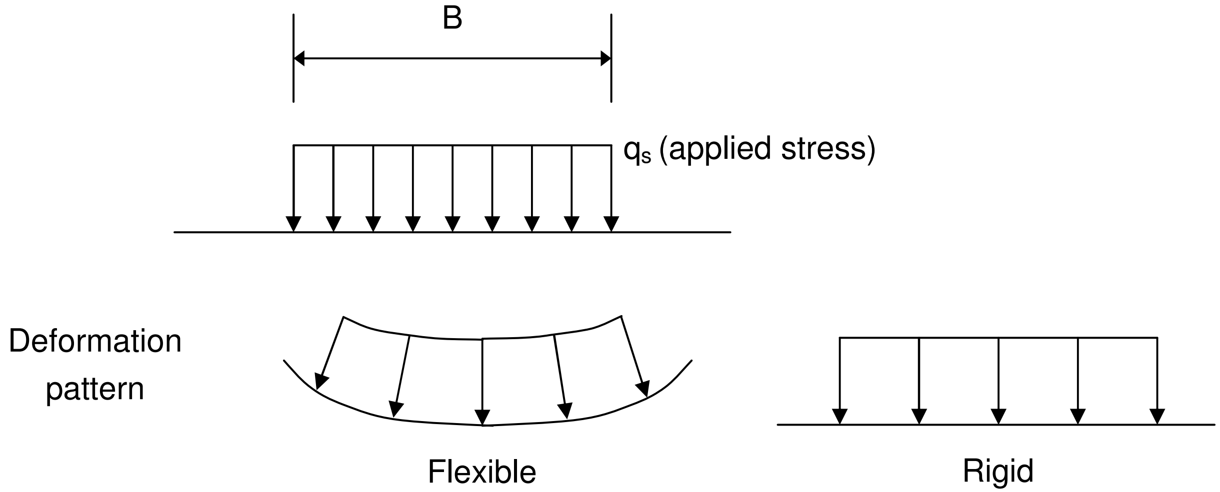
2.2 Rate-Independent Settlements

(1) Elastic Theory



- For D_f(=depth to the bottom of foundation level) = 0 and assuming soil as an elastic material,

$$S_i = \frac{\text{Immediate}}{\text{Settlement}} = q\,\frac{B}{E}\,I_p\,(1-\nu^2) \qquad \text{- (1)}$$

E, ν : "elastic" parameter for soil

ν = 0.5 for undrained loading

ν = 0.2~0.4 for drained loading

I_p : shape and rigidity factor

i) On an elastic half space of unlimited depth : Table 1 in p.2-17

ii) On an elastic half space of limited depth over a rigid base : Table 1 in p.2
18

- For $D_f > 0$ (stiff to hard fine grained soil for rectangular footing)

  → Fig. 11-9 in p.2-20

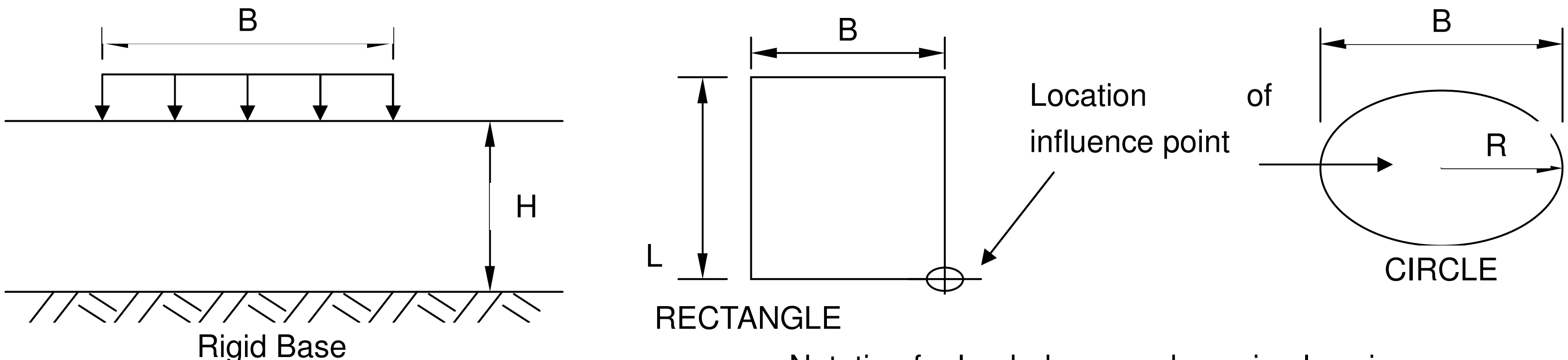
1. Get C_s. (→ I_p)
2. Calculate settlement for $D_f = 0$, $\delta_{v0} = \dfrac{0.75\, q_s\, B\, C_s \sqrt{L/B}}{E_s}$.
3. Obtain δ_{vd}/δ_{v0} ratio from chart, with $D/\sqrt{BL}$ and L/B.
4. then, get δ_{vd} = settlement at deep footing ($\delta_{vd} < \delta_{v0}$).

Table 1. Shape and rigidity factors I_p for calculating settlements of points on loaded area at the surface of an elastic half-space

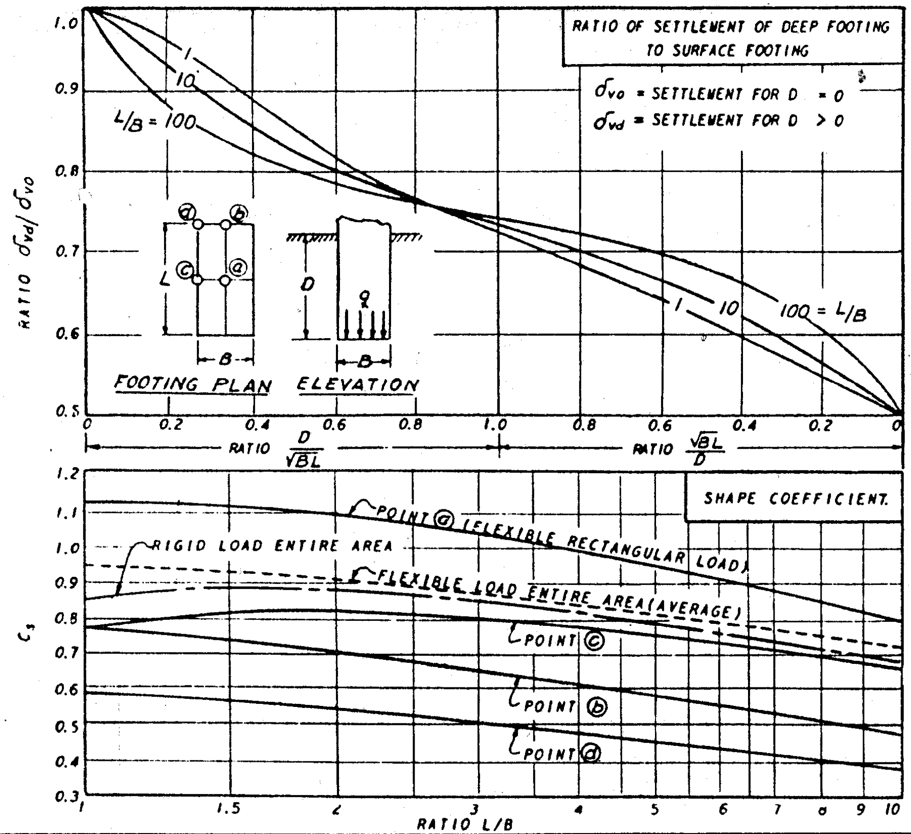
Shape and Rigidity Factors I for Loaded Areas on an Elastic Half-Space of Infinite Depth				
Shape and Rigidity	Center	Corner	Edge/Middle Of Long Side	Average
Circle (flexible)	1.00		0.64	0.85
Circle (rigid)	0.79		0.79	0.79
Square (flexible)	1.12	0.56	0.76	0.95
Square (rigid)	0.82	0.82	0.82	0.82
Rectangle (flexible) Length/width				
2	1.53	0.76	1.12	1.30
5	2.10	1.05	1.68	1.82
10	2.56	1.28	2.10	2.24
Rectangle (rigid) Length/width				
2	1.12	1.12	1.12	1.12
5	1.60	1.60	1.60	1.60
10	2.00	2.00	2.00	2.00

Shape and Rigidity Factor I for Loaded Areas on an Elastic Half-Space of Limited Depth Over a Rigid Base

H/B	Center of Rigid Circular Area Diameter = B	Corner of Flexible Rectangular Area				
		L/B = 1	L/B = 2	L/B = 5	L/B = 10	(strip) L/B=∞
For $\nu = 0.50$						
0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.14	0.05	0.04	0.04	0.04	0.04
1.0	0.35	0.15	0.12	0.10	0.10	0.10
1.5	0.48	0.23	0.22	0.18	0.18	0.18
2.0	0.54	0.29	0.29	0.27	0.26	0.26
3.0	0.62	0.36	0.40	0.39	0.38	0.37
5.0	0.69	0.44	0.52	0.55	0.54	0.52
10.0	0.74	0.48	0.64	0.76	0.77	0.73
For $\nu = 0.33$						
0.0	0.00	0.00	0.00	0.00	0.00	0.00
0.5	0.20	0.09	0.08	0.08	0.08	0.08
1.0	0.40	0.19	0.18	0.16	0.16	0.16
1.5	0.51	0.27	0.28	0.25	0.25	0.25
2.0	0.57	0.32	0.34	0.34	0.34	0.34
3.0	0.64	0.38	0.44	0.46	0.45	0.45
5.0	0.70	0.46	0.56	0.60	0.61	0.61
10.0	0.74	0.49	0.66	0.80	0.82	0.81



Notation for loaded areas, shown in plan view



STIFF TO HARD FINE GRAINED SOILS

(MODULUS E_s IS CONSTANT WITH DEPTH)

SETTLEMENT FOR D = 0 (δ_{vo}):

$$\delta_{vo} = \frac{.75\, q\, B\, C_s \sqrt{L/B}}{E_s}$$

AND SINCE $E_s = .72\ K_{v1}$ (TONS AND FT. UNITS)

$$\delta_{vo} = \frac{1.05\, q\, B\, C_s \sqrt{L/B}}{K_{v1}}$$

SETTLEMENT FOR D > 0 (δ_{vd}):
COMPUTE δ_{vo} AND OBTAIN RATIO OF δ_{vd}/δ_{vo} FROM UPPER DIAGRAM.

POSITION OF GROUND WATER LEVEL DOES NOT AFFECT IMMEDIATE SETTLEMENT UNLESS SOIL IS PERMITTED TO SWELL. FOUNDATION MATERIAL IS ASSUMED UNIFORM TO A DEPTH OF AT LEAST 1.5B BELOW BASE OF FOOTING.

CONVERSION OF CONSTANT MODULUS E_s TO EQUIVALENT MODULUS E_s VARYING LINERALY WITH DEPTH FOR APPLICATION TO LATERALLY LOADED PILES IN STIFF TO HARD CLAYS.

PROCEDURE:

1. GIVEN A VALUE OF CONSTANT E_s. ASSUME A VALUE OF f.
2. COMPUTE DEPTH z TO POINT OF ZERO DEFLECTION OF PILE FROM FIGS. 13-4 OR 13-5.
3. RECOMPUTE f SO THAT VALUE OF $fz = 2E_s$.
4. RECOMPUTE DEPTH z WITH REVISED f AND ALTER f AGAIN SO THAT $fz = 2E_s$.

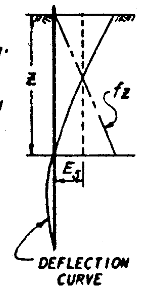


FIGURE 11-9
Immediate Settlement of Isolated Footings on Stiff to Hard Fine Grained Soils

● Settlements on Soils

- Settlements

  i) Immediate → Time independent settlements

  ii) Primary consolidation

  iii) Secondary compression

  ii) and iii) → Time dependent settlements

- Sands → Immediate

  → Settlement due to 2-D or 3-D deformations and volume compression ($\nu < 0.5$)

  Primary consolidation

  → Settlement due to volume compression related to dissipation of pore pressure

  → Negligible due to low compressibility of sands and empirical implementation on existing immediate settlement computation method with $\nu < 0.5$

  Secondary compression → creep

- Clays $\rightarrow$ Immediate (undrained) $\leftarrow$ 2-D or 3-D conditions

  Primary consolidation

  Secondary compression

(2) Immediate Settlements in Cohesive Soils

- 2-D or 3-D condition → very important
- 1-D condition → negligible (can be considered in unsaturated soils)
- Non-linearity is a key point to estimate settlements.

<D'Appolonia, et. al. (1971)>

- Immediate settlements that account for local yielding

  (Significant in $(FS)_{bc} < 3 \Rightarrow$ increasing settlement.)

- Based on finite element analysis with an elasto-plastic soil model.

  SR (Settlement Ratio) = $\dfrac{\delta_v}{\delta_c}$

  δ_v : elastic component of settlement (no yielding) $= qB\dfrac{(1-\nu)^2}{E_u} I_p$

  δ_c : immediate settlement corrected to allow for partial yield conditions.

- Method

  ∗ Strictly applicable to strip loads.

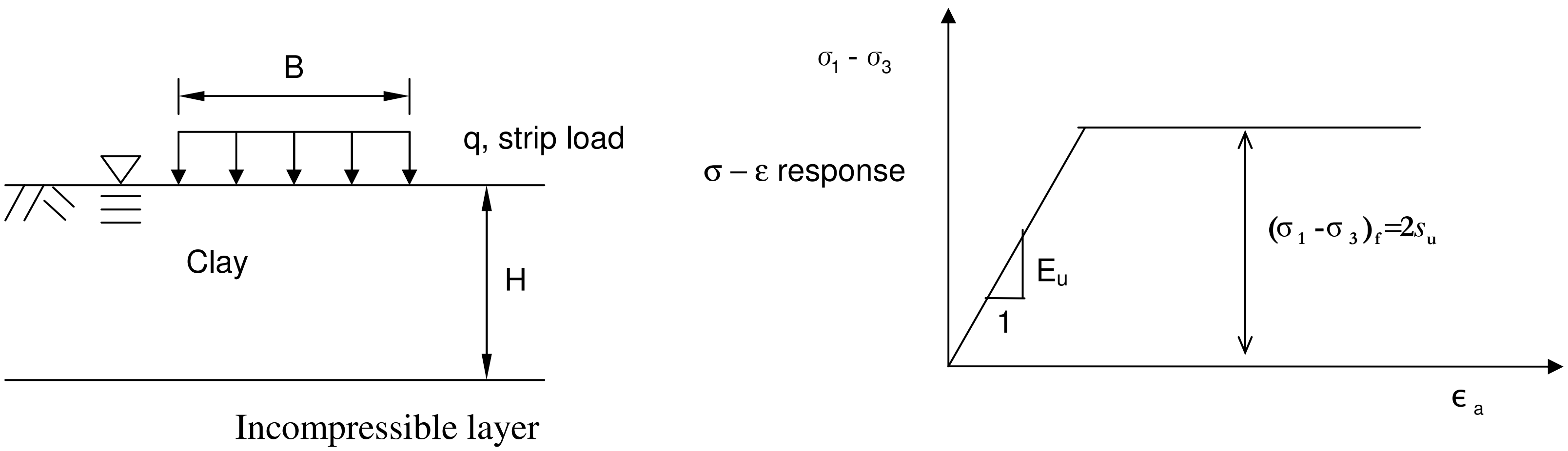
  (Conservative for loads of limited extent)

  ∗ Accounts for

  - initial shear stress ratio.
  - thickness of clay layer.
  - applied stress level.

- How was the method developed?

i) D'Appolonia (1971) performed numerical simulations of soil profile :



⇒ Computed $\delta_{FE.}$ (corresponds to δ_c) → settlement under center line of footing.

ii) Calculate $\delta_E (= \delta_v)$ based on elastic theory using same values of E_u that D'Appol. used in finite element simulations.

iii) Calculate SR(= δ_v / δ_c) using (1) and (2) for various conditions.

a) H/B

b) initial shear stress ratio ($f = \left(\dfrac{\sigma'_{v0} - \sigma'_{h0}}{2}\right) \Big/ s_u = \dfrac{(1 - K_0)}{2 s_u / \sigma'_{v0}}$)

c) Applied stress $= q/q_u = 1/F.S.$

q_u : ultimate bearing capacity

→ DM 7.1 – 216 in p.2-24

(f ↑, H/B ↑, q/q_{ult} ↑. ⇒ SR ↓ (⇒ increase settlement))

- Approach to get δ_c

① Calculate f if we know K_o and s_u/σ'_{V0} .***or*** Estimate f from Fig. 4b (DM 7.1-217) in p.2-26.

② Obtain E_u

a) Using Table 2. (DM 7.1-215) in p.2-25.

b) Run high quality tests.

③ Perform undrained ultimate bearing capacity analysis.

$$q_{ult} = 5.53c = 5.53s_u$$

→ Ratio of q/q_{ult}

④ For given H/B, q/q_{ult} and f, obtain SR from Fig. 4a (DM 7.1-216) in p.2-24.

⑤ Calculate $\delta_v = q \frac{B}{E_u} I_p (1 - \nu^2)$, where E_u is consistent with that found in step 2

⑥ $$\delta_c = \frac{\delta_v}{SR}$$

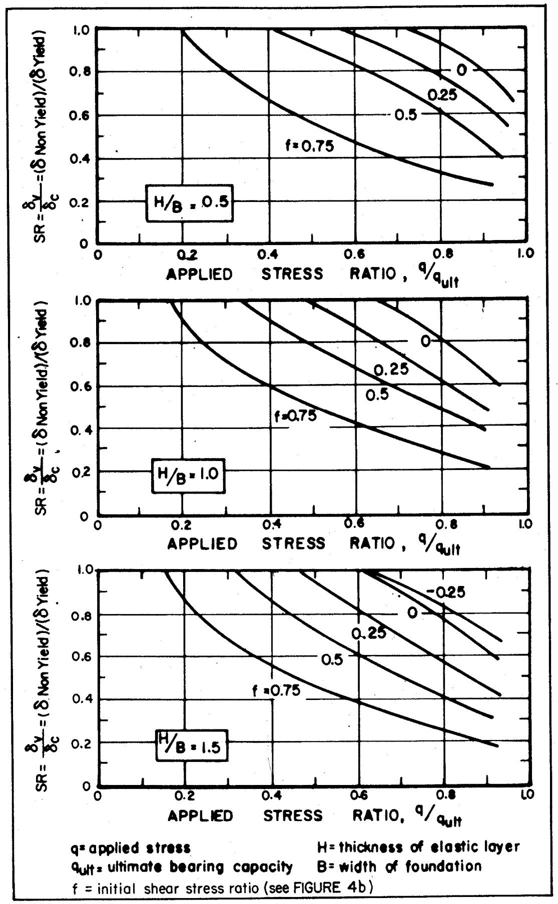


FIGURE 4a
Relationship Between Settlement Ratio and Applied Stress Ratio for Strip Foundation on Homogeneous Isotropic Layer

7.1-216

Table 2

Relationship Between Undrained Modulus and Overconsolidation Ratio

OCR	E_u/s_u		
	PI < 30	30 < PI < 50	PI > 50
< 3	600	300	125
3 ~ 5	400	200	75
> 5	150	75	50

- OCR = Overconsolidation ratio
- s_u = Undrained shear strength
- PI = Plastic index

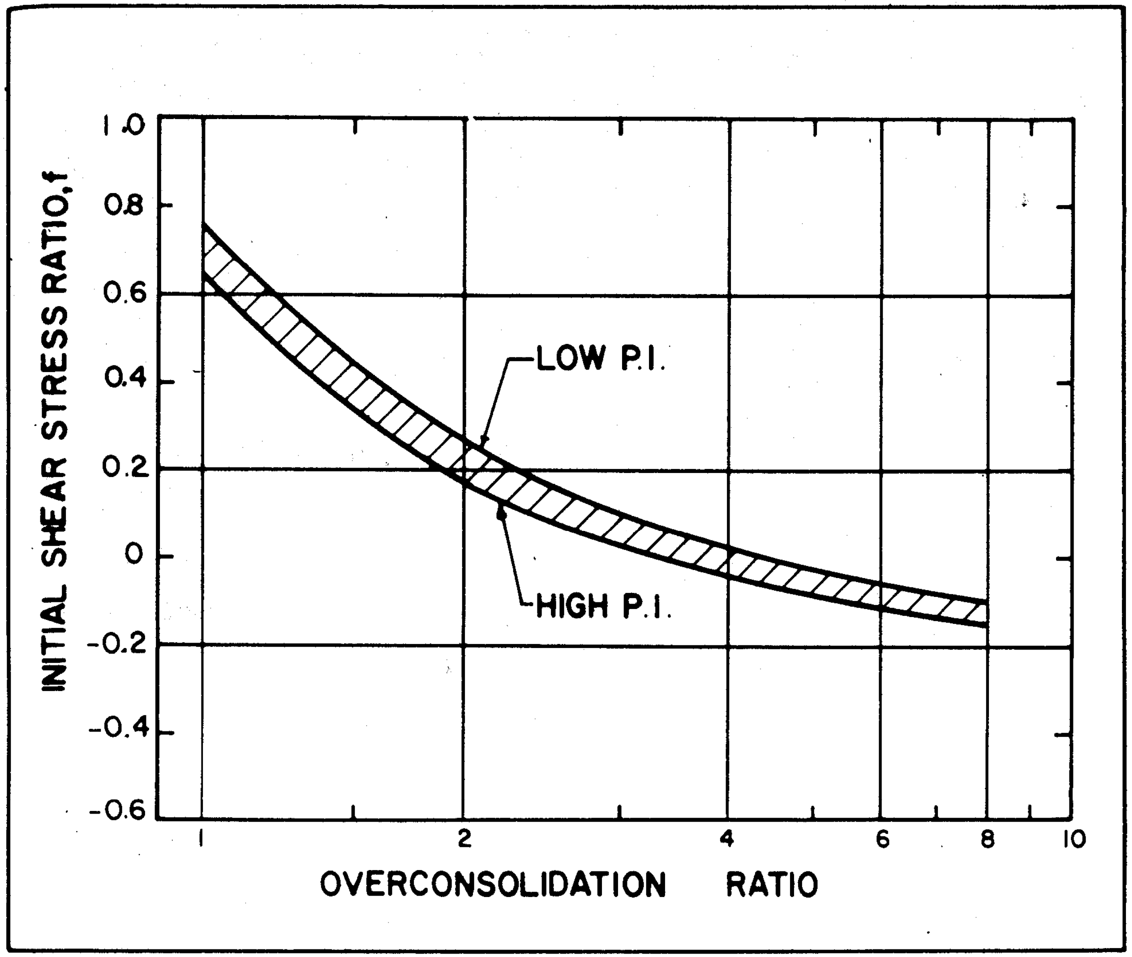


FIGURE 4b
Relationship Between Initial Shear Stress and Overconsolidation Ratio

- Comments

i) When H/B > 1.5, influence of the greater depth of clay is small.

→ Use chart for H/B=1.5 for all values.

ii) Settlements are at the centerline of strip footing.

iii) Unless the soil is highly anisotropic($E_{u(horizontal)} > E_{u(vertical)}$), the results of isotropic analysis are good enough to estimate δ_c.

iv) Influence of strength nonhomogeneity on settlment is relatively small, compared to f.

v) Immediate settlements in clays are important when clay is highly plastic or organic.

$\frac{E_u}{s_u}$ decreases with increasing plasticity.

vi) When immediate settlements are significant, then creep movements are likely to occur.

vii) Surface footing only.→ Need modification for embedded footing.

(3) Immediate Settlements in cohesionless soils.

- Elastic theory
- SPT
- Plate Load Test
- CPT

i) Elastic Theory

- Same procedure as before

→ Problem : E value?

↓

Typical, $E = K(\sigma'_3)^n$, n ≈ 1/2, K=constant

→ To obtain E in lab. testing,

we need to know : Insitu D_r (relative density) + (water content)

(Evaluate D_r from empirical correlation with SPT and CPT results.)

→Find $\gamma_{d\max}$ and $\gamma_{d\min}$.

→Reconstitute sample to appropriate γ_d (corresponding to D_r) and water content.

→Run shear test.

ii) SPT

- <u>Meyerhof</u>

For B ≤ 4' , $S_e = \dfrac{4q_s}{N}$ (q_s in klb/ft_2)

For B> 4' , $S_e = \dfrac{KC_d q_s}{N\left(\dfrac{B+1}{B}\right)^2}$ (B in feet, q_s in klb/ft^2)

K ≡ constant

= 12 (original value → very conservative)

= 8 (max. settlement)

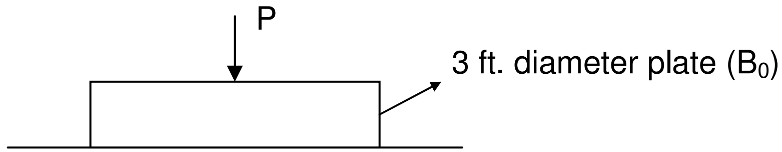
= 4 (average settlement)

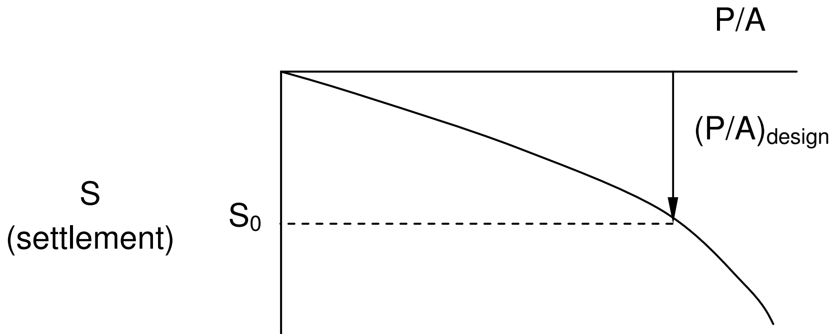
C_d ≡ correction factor for embedment depth,(D_f=0 → C_d=1, D_f=B → C_d=0.75)

q_s ≡ average pressure

N ≡ average N value in the stressed zone (≈ 0~2B)

iii) Plate Load Test





$$\frac{S}{S_o} = \frac{4}{\left(1+\frac{B_0}{B}\right)^2} \Rightarrow S = \frac{4S_o}{\left(1+\frac{B_0}{B}\right)^2}$$

B/B_0 > 4~5 → not a good results,

sometimes actual settlements ≈ 3 x estimated settlements.

- Bonds recommends,

$$\frac{(S/B)}{(S_0/B_0)} = \left(\frac{B}{B_0}\right)^n$$

n = 0.03 to 0.05 for clay

0.08 to 0.10 for sandy clay

0.40 to 0.50 for dense sand

0.25 to 0.35 for medium dense sand

0.20 to 0.25 for loose sand

- Problem