CE464 Ground Improvement

Fall 2016

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Compaction - Part 2

Benefit of replacing the upper part of weak soil with compacted granular soil

GROUND IMPROVEMENT

Compaction is usually an economical method of improving the bearing capacity of site soils.

It may be accomplished by excavating to some depth, then carefully backfilling in controlled lift thicknesses, each of which is compacted with the appropriate compaction equipment.

Purpose of compaction is to :

- Decrease compressibility and future settlements
- Increase shear strength
- Increase bearing capacity
- Decrease permeability
- Replacement of weak soil with a compacted stiffer layer can reduce vertical stresses with depth underneath the loaded area

Elastic modulus (Young's modulus, or stiffness) of Compacted Soil

Stiffness of granular and stiff (overconsolidated) cohesive soils based on N_{60} values are approximately given as follows (Stroud 1989):

E' (MPa) = $2 \times N_{60}$ (overconsolidated sands and gravels)

E' (MPa) = N_{60} (normally consolidated sands)

E' (MPa) = $(0.7 - 0.9) \times N_{60}$ (stiff overconsolidated clays: for plastic, I_p=50%, and less plastic, I_p=15% clays, respectively)

[Note that: $E' / E_u = (1+\upsilon) / (1+\upsilon_u)$]

υ = Poisson's ratio

These values are roughly for the strain levels encountered at allowable foundation loads. At very small strain values they are much higher.

Modulus of elasticity, E_s

In sandy soils

$$\frac{E_s}{p_a} = \alpha N_{60}$$

p_a = atmospheric pressure ≈ 100 kN/m^2

$$\alpha = \begin{cases} 5 \text{ for sands with fines} \\ 10 \text{ for clean normally consolidated sand} \\ 15 \text{ for clean overconsolidated sand} \end{cases}$$

Modulus of elasticity, E_s (E_u)

In clays:

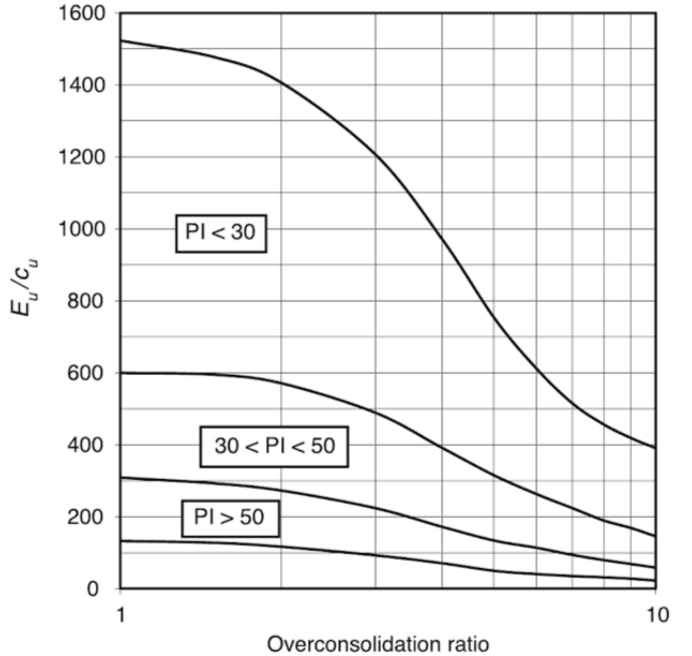
$$\frac{E_u}{c_u} = ratio$$

c_u= undrained shear strength

Plasticity index	*ratio*				
	OCR = 1	OCR = 2	OCR = 3	OCR = 4	OCR = 5
< 30	1500–600	1380–500	1200–580	950–380	730–300
30 to 50	600–300	550–270	580–220	380–180	300–150
> 50	300–150	270–120	220–100	180–90	150–75

[a]Interpolated from Duncan and Buchignani (1976)

$$\frac{E'}{E_u} = \frac{(1+v)}{(1+v_u)} = \frac{(1+0.25)}{(1+0.45)} = 0.862$$



Duncan and Buchignani (1976)

Table 3.16 Typical values of E_u for clays

Clay	E_u (MPa)
Very soft clay	0.5–5
Soft clay	5–20
Medium clay	20–50
Stiff clay, silty clay	50–100
Sandy clay	25–200
Clay shale	100–200

After U.S. Army (1994) and Bowles (1986)

Table 3.17 Typical values of Poisson's ratio

Material	Poisson's ratio
Saturated clays (undrained)	0.5
Saturated clays (drained)	0.2–0.4
Dense sand	0.3–0.4
Loose sand	0.1–0.3
Loess	0.1–0.3
Ice	0.36
Aluminum	0.35
Steel	0.29
Concrete	0.15

Bowles (1986), Kulhawy and Mayne (1990), and Lambe and Whitman (1979)

Correlations of Soil and Rock Properties in Geotechnical Engineering, 2015 (Ameratunga, Sivakugan, Das)

Typical values of Young's modulus for granular material (MPa) (based on Obrzud & Truty 2012 complied from Kezdi 1974 and Prat et al. 1995)

USCS	Description	Loose	Medium	Dense
GW, SW	Gravels/Sand well-graded	30-80	80-160	160-320
SP	Sand, uniform	10-30	30-50	50-80
GM , SM	Sand/Gravel silty	7-12	12-20	20-30

Typical values of Young's modulus for cohessive material (MPa) (based on Obrzud & Truty 2012 compiled from Kezdi 1974 and Prat et al. 1995)

USCS	Description	Very soft to soft	Medium	Stiff to very stiff	Hard
ML	Silts with slight plasticity	2.5 - 8	10 - 15	15 -40	40 - 80
ML, CL	Silts with low plasticity	1.5 - 6	6 -10	10 - 30	30 -60
CL	Clays with low-medium plasticity	0.5 - 5	5 -8	8 - 30	30 - 70
CH	Clays with high plasticity	0.35 - 4	4 -7	7 - 20	20 - 32
OL	Organic silts	-	0.5 -5	-	-
OH	Organic clays	-	0.5 -4	-	-

http://geotechdata.info/parameter/soil-elastic-young-modulus.html

Table C.2 Typical values of modulus of elasticity (E_S) for different types of soils

Type of Soil	E_s (N/mm^2)
Clay	
Very soft	2–15
Soft	5–25
Medium	15–50
Hard	50–100
Sandy	25–250
Glacial till	
Loose	10–153
Dense	144–720
Very dense	478–1,440
Loess	14–57
Sand	
Silty	7–21
Loose	10–24
Dense	48–81
Sand and gravel	
Loose	48–148
Dense	96–192
Shale	144–14,400
Silt	2–20

Increase in Vertical stresses **($\Delta\sigma_v$')** : Stiff layer over a soft layer:
the effect of replacing the upper part of weak soil with a stiff layer will be to reduce the stresses ($\Delta\sigma_v$') in the lower layer (hence reduce consolidation settlement)

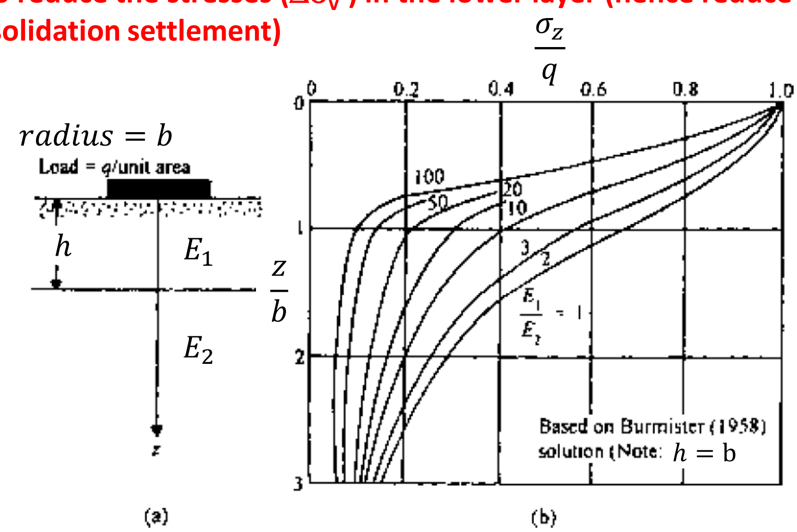


Figure 3.29 (*a*) Uniformly loaded circular area in a two-layered soil $E_1 > E_2$ and (*b*) Vertical stress below the centerline of a uniformly loaded circular area.

Vertical stress increase due to a Uniform Flexible **Strip** Foundation on a Two-layered Soil Profile

Boussinesq stress distribution theory is for homogeneous, elastic, isotropic, semi-infinite soil. Elastic solutions for multi-layered soils are studied earliest by Burmister (1943) and by Fox (1948)

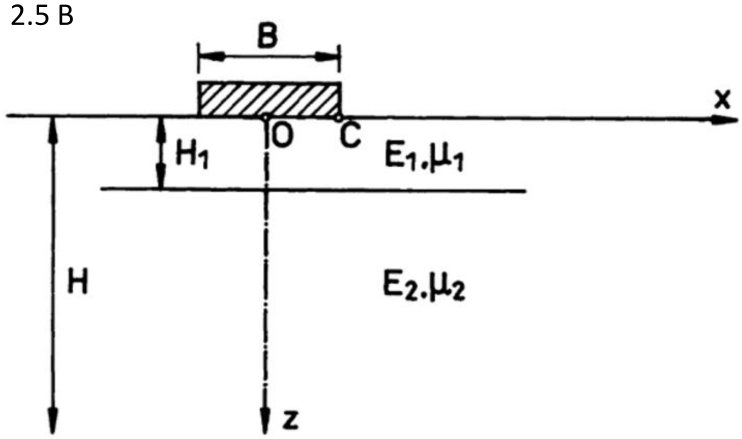
Mitchell and Gardner (1971) studied load bearing fill problem by finite element method. They also confirmed that soil layer of a relatively high deformation modulus (E), placed over weak foundation soil is effective in reducing the magnitude of the stress induced in the weak subsoil.

Stresses and Displacements for Shallow Foundations, Milovic (2013)

Flexible **strip** foundation of width B = 1.0 m

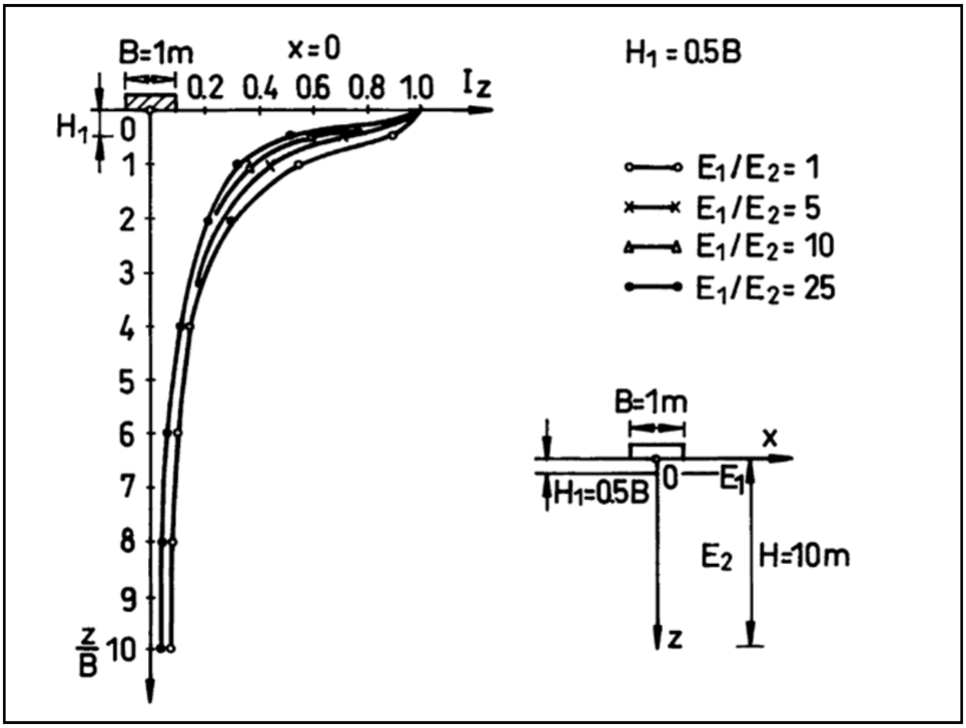
H_1 = 0.5 B
1 B
1.5 B
2 B
2.5 B

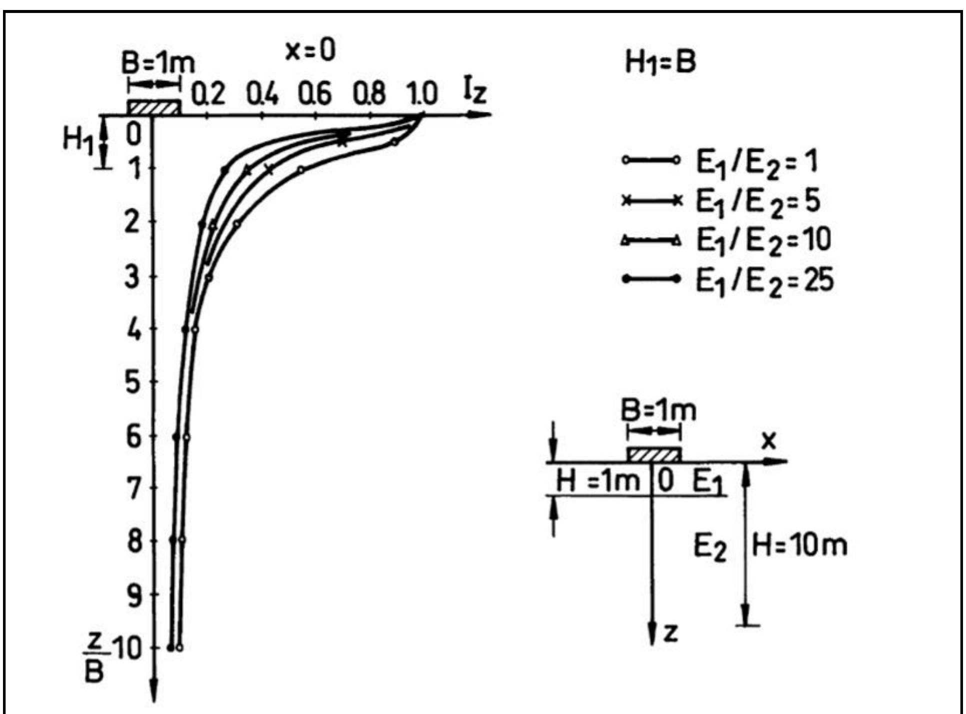
E_1 / E_2 = 1
5
10
25

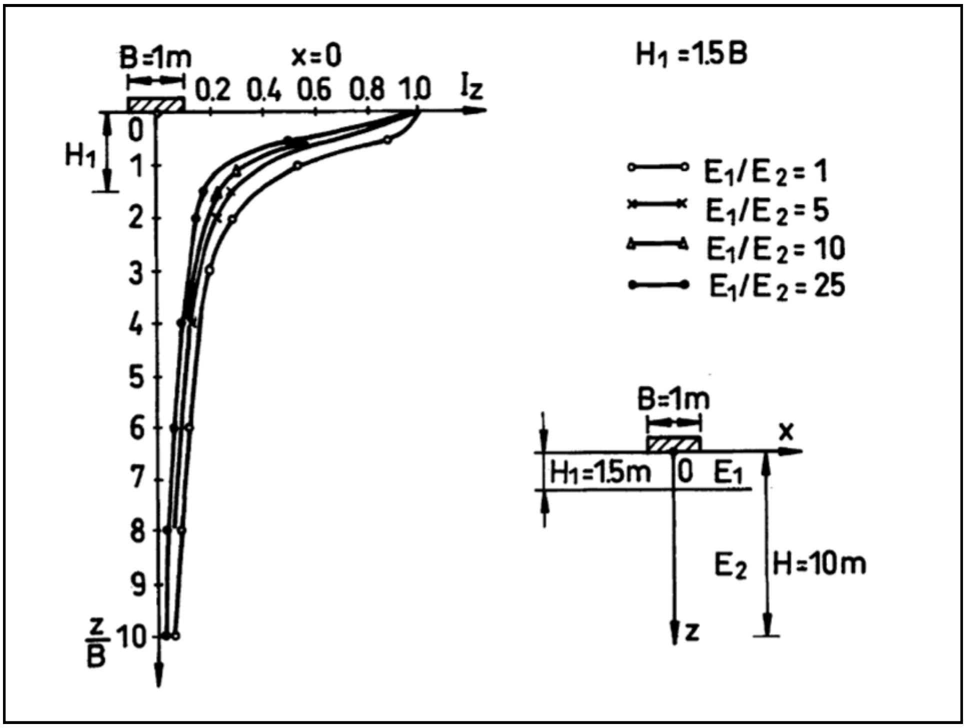


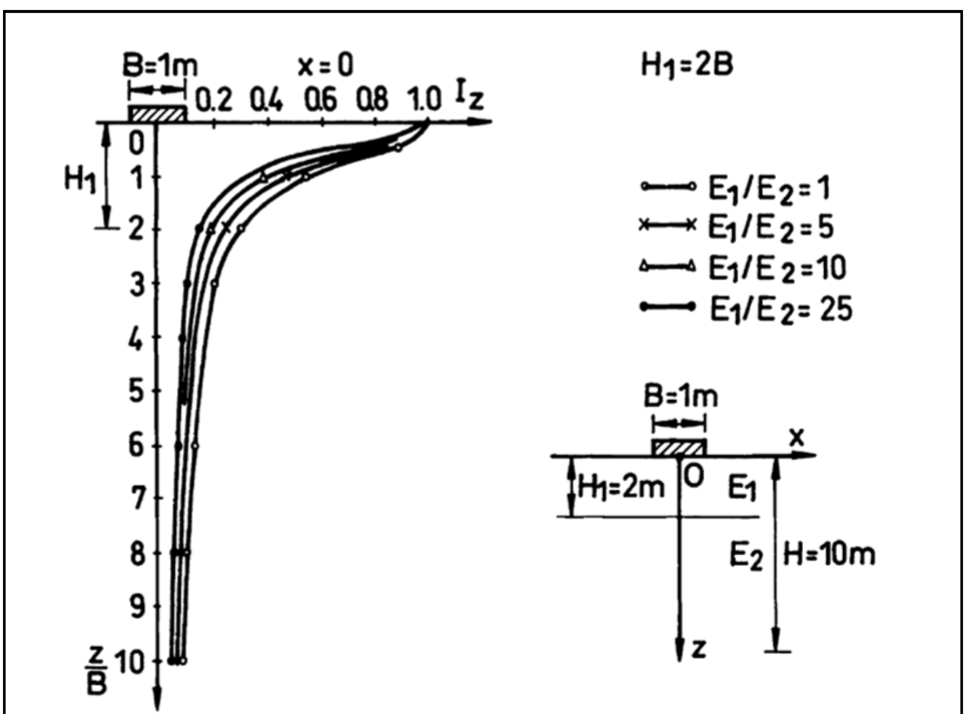
$$(\Delta\sigma_v') = \sigma_z = p \cdot I_z$$

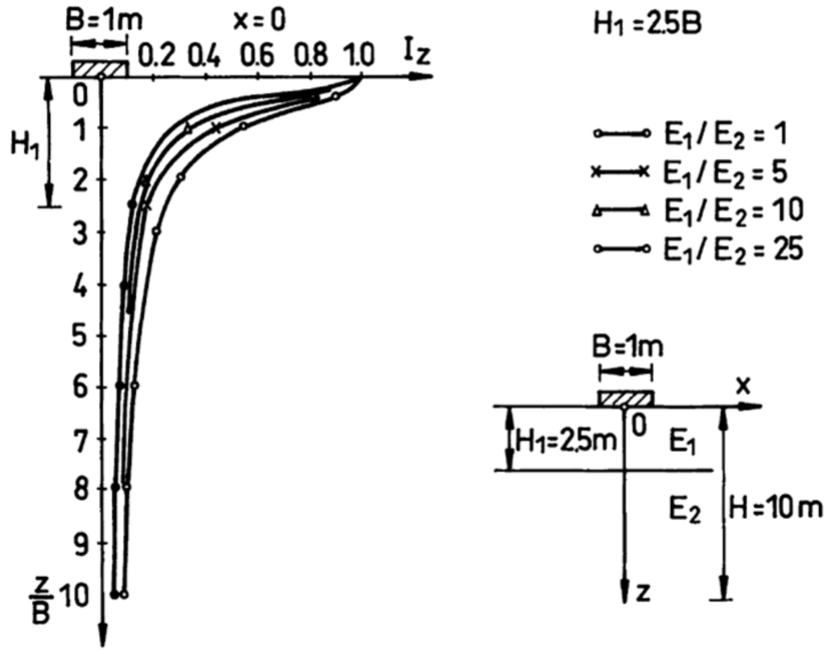
where p = the applied load and I_z = the dimensionless coefficient.


B=1m
x=0
H1 = 0.5B
0.2 0.4 0.6 0.8 1.0 Iz
H1
0
1
2
3
4
5
6
7
8
9
z/B 10
E1/E2 = 1
E1/E2 = 5
E1/E2 = 10
E1/E2 = 25
B=1m
x
H1=0.5B
0
E1
E2
H=10m
z


B=1m
x=0
H1=B
0.2 0.4 0.6 0.8 1.0 Iz
H1
0
1
2
3
4
5
6
7
8
9
z/B 10
E1/E2 = 1
E1/E2 = 5
E1/E2 = 10
E1/E2 = 25
B=1m
x
H =1m
0
E1
E2
H=10m
z


H1 =1.5B
B=1m
x=0
0.2 0.4 0.6 0.8 1.0 Iz
H1
z/B
E1/E2=1
E1/E2=5
E1/E2=10
E1/E2=25
B=1m
x
H1=1.5m
0
E1
E2
H=10m
z

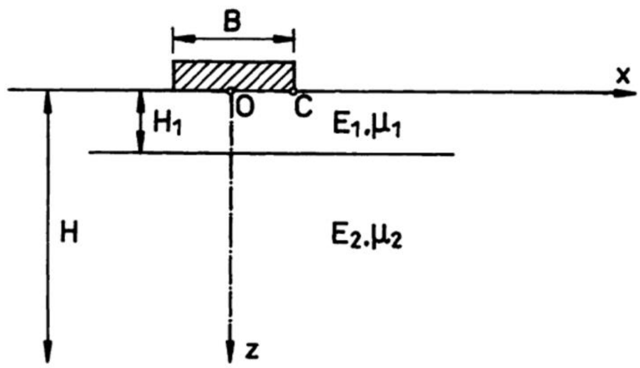

H1=2B
B=1m
x=0
0.2 0.4 0.6 0.8 1.0 Iz
H1
z/B
E1/E2=1
E1/E2=5
E1/E2=10
E1/E2=25
B=1m
x
H1=2m
O
E1
E2
H=10m
z



Immediate settlement of the flexible **strip** foundation, two-layered system

$$w = \frac{pB}{E_2} I_w$$

where p = the applied load; B = the width of the foundation; E_2 = the modulus of elasticity of the lower layer and I_w = the dimensionless coefficient.



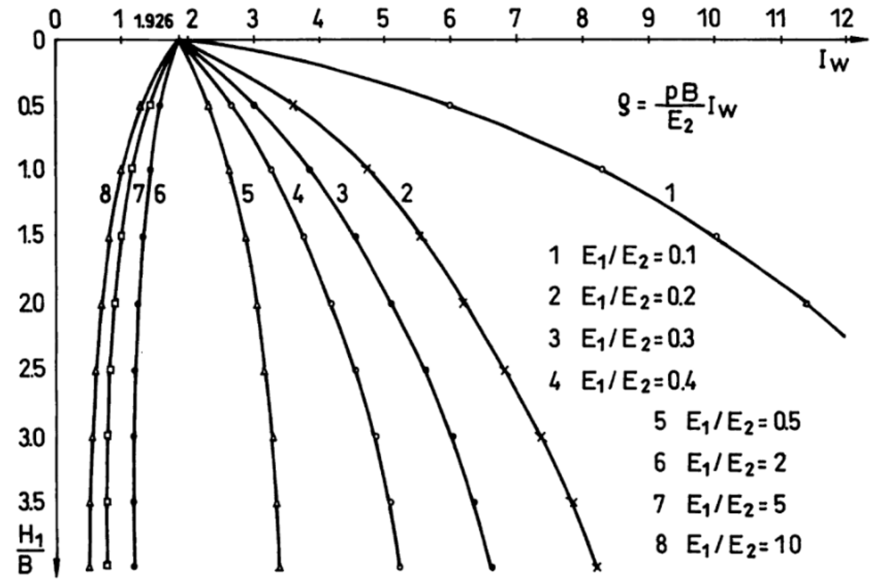


Fig. 2.24 Coefficients I_w; point 0, $\mu = 0.30$.

Coefficients I_w; flexible strip foundation.

$x = 0$ $\mu_1 = \mu_2 = 0{,}30$

$\frac{H_1}{B}$ \ $\frac{E_1}{E_2}$	0,1	0,2	0,5	2	5	10
1,0	8,251	4,816	2,650	1,496	1,204	1,079
2,0	11,413	6,217	3,027	1,370	0,960	0,790
3,0	13,283	7,040	3,232	1,280	0,807	0,612
5,0	15,792	8,094	3,498	1,135	0,579	0,401

Coefficients I_w; flexible strip foundation.

$x = 0$ $\mu_1 = \mu_2 = 0{,}49$

$\frac{H_1}{B}$ \ $\frac{E_1}{E_2}$	0,1	0,2	0,5	2	5	10
1,0	5,427	3,270	1,939	1,209	0,941	0,805
2,0	8,315	4,554	2,258	1,051	0,703	0,539
3,0	10,073	5,334	2,451	0,962	0,562	0,370
5,0	12,214	6,272	2,682	0,850	0,421	0,247

Immediate settlement of the rigid strip foundation, two-layered system

2.2 RIGID STRIP FOUNDATION IN A TWO-LAYERED SYSTEM

In the study of stresses and displacements of the perfectly rigid strip foundation, resting on elastic two-layer soil, the finite element method was used. The dimensionless parameters I_w for the settlement calculation have been determined for the thickness H_1 of the upper layer H_1=B, 2B, 3B and 5B and for several values of the ratio E_1/E_2.

Coefficients I_w; rigid strip foundation.

$\frac{x}{B}$ \ $\frac{E_1}{E_2}$	0,1	0,2	0,5	2,0	5,0	10,0
H_1 = B			$\mu_1 = \mu_2 = 0{,}30$			
0,00	6,927	4,158	2,410	1,451	1,197	1,065
0,50	6,923	4,156	2,400	1,456	1,200	1,068
0,75	3,154	2,171	1,552	1,207	1,070-	0,996
1,00	1,511	1,284	1,121	1,029	0,968	0,925
H_1 = 2B			$\mu_1 = \mu_2 = 0{,}30$			
0,00	9,866	5,457	2,715	1,296	0,939	0,778
0,50	9,864	5,454	2,711	1,298	0,939	0,779
0,75	5,651	3,301	1,841	1,063	0,838	0,728
1,00	3,346	2,119	1,341	0,921	0,773	0,687
H_1 = 3B			$\mu_1 = \mu_2 = 0{,}30$			
0,00	11,724	6,260	2,909	1,191	0,781	0,600
0,50	11,721	6,257	2,901	1,194	0,779	0,598
0,75	7,443	4,116	2,056	0,966	0,675	0,544
1,00	4,843	2,789	1,510	0,830	0,605	0,506
H_1 = 5B			$\mu_1 = \mu_2 = 0{,}30$			
0,00	14,303	7,374	3,179	1,059	0,585	0,401
0,50	14,300	7,372	3,178	1,064	0,583	0,404
0,75	9,926	5,190	2,307	0,845	0,495	0,352
1,00	7,335	3,881	1,779	0,700	0,431	0,329

Coefficients I_w; rigid strip foundation.

$\frac{x}{B}$ \ $\frac{E_1}{E_2}$	0,1	0,2	0,5	2,0	5,0	10,0
	$H_1 = B$			$\mu_1 = \mu_2 = 0,49$		
0,00	4,261	2,671	1,661	1,112	0,882	0,772
0,50	4,259	2,668	1,658	1,109	0,879	0,771
0,75	1,201	1,059	0,969	0,867	0,770	0,703
1.00	1,108	0,782	0,716	0,689	0,664	0,518
	$H_1 = 2B$			$\mu_1 = \mu_2 = 0,49$		
0,00	6,812	3,779	1,941	0,959	0,652	0,520
0,50	6,808	3,776	1,938	0,954	0,659	0,516
0,75	3,111	1,905	1,168	0,754	0,571	0,472
1,00	1,236	0,921	0,749	0,621	0,510	0,374
	$H_1 = 3B$			$\mu_1 = \mu_2 = 0,49$		
0,00	8,496	4,530	2,128	0,873	0,530	0,358
0,50	8,494	4,530	2,128	0,869	0,526	0,355
0,75	4,508	2,540	1,330	0,671	0,447	0,315
1,00	2,318	1,422	0,875	0,559	0,400	0,290
	$H_1 = 5B$			$\mu_1 = \mu_2 = 0,49$		
0,00	10,571	5,441	2,349	0,769	0,385	0,234
0,50	10,572	5,442	2,348	0,764	0,381	0,234
0,75	6,549	3,483	1,549	0,565	0,307	0,190
1,00	4,311	2,351	1,103	0,448	0,246	0,161

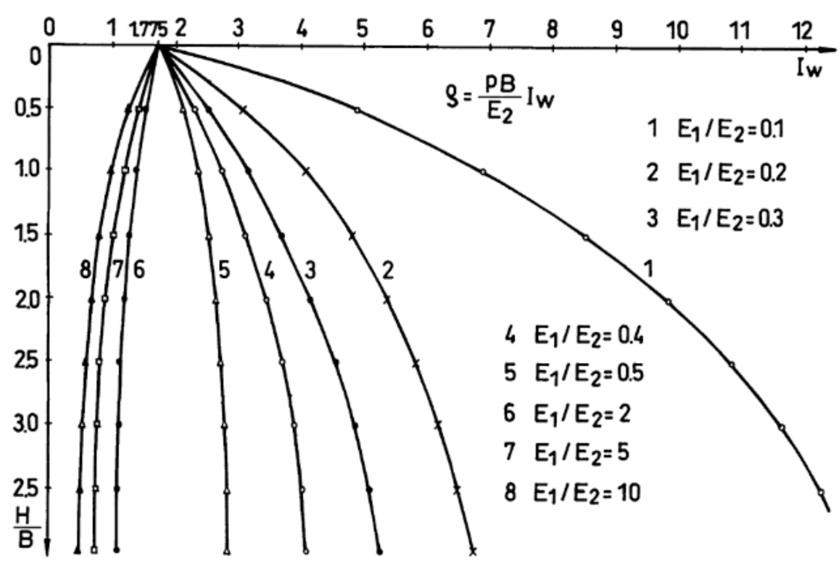


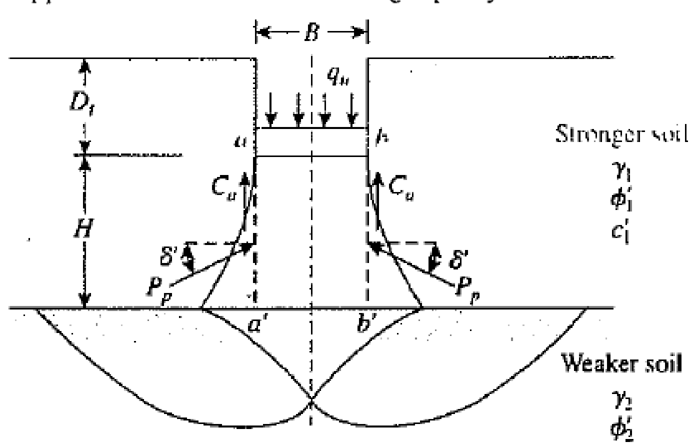
Fig. 2.32 Coefficients I_w; rigid strip foundation.

Stiff layer over a soft layer:

the effect of replacing the upper part of weak soil with the stiff layer will be also to increase bearing capacity

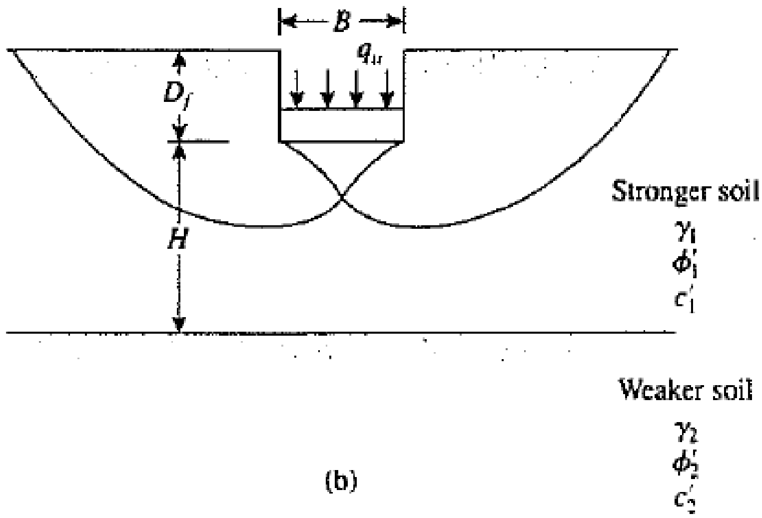
At ultimate load per unit area (q_u), the failure surface in soil will be as shown in the figure. If the depth H is relatively small compared with the foundation width B, a punching shear failure will occur in the top soil layer, followed by a general shear failure in the bottom soil layer.

However, if the depth H is relatively large, then the failure surface will be completely located in the top soil layer, which is the upper limit for the ultimate bearing capacity.



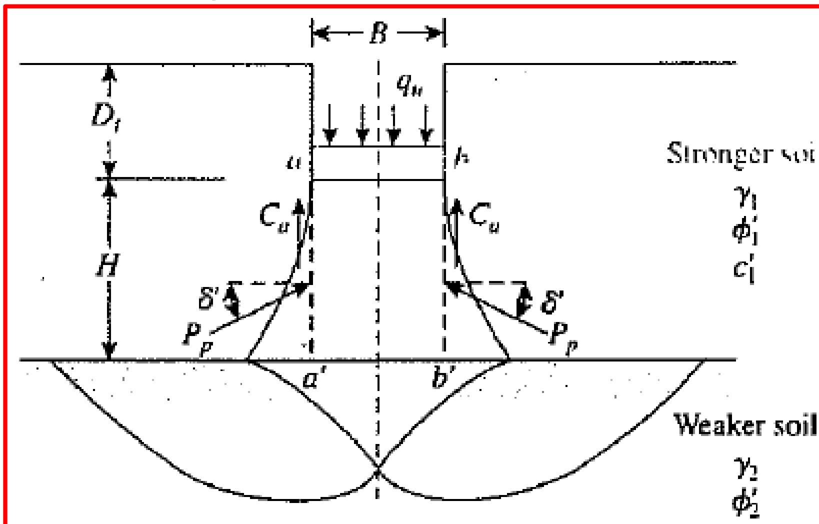
(a)

However, if the depth H is relatively large, then the failure surface will be completely located in the top soil layer, which is the upper limit for the ultimate bearing capacity.



(b)

At ultimate load per unit area (q_u), the failure surface in soil will be as shown in the figure. If the depth H is relatively small compared with the foundation width B, a punching shear failure will occur in the top soil layer, followed by a general shear failure in the bottom soil layer.



(a)

The ultimate bearing capacity for this problem be given as

$$q_u = q_b + \frac{2(C_a + P_p \sin\delta')}{B} - \gamma_1 H$$

where

B = width of the foundation
C_a = adhesive force
P_p = passive force per unit length of the faces aa' and bb'
q_b = bearing capacity of the bottom soil layer
δ' = inclination of the passive force P_p with the horizontal

Note that, in Eq. (4.9),

$$C_a = c'_a H$$

where c'_a = adhesion.

This equation can be simplified to the form:

$$q_u = q_b + \frac{2c'_a H}{B} + \gamma_1 H^2\left(1 + \frac{2D_f}{H}\right)\frac{K_{pH}\tan\delta'}{B} - \gamma_1 H$$

where K_{pH} = horizontal component of passive earth pressure coefficient.

Then to this form:

$$q_u = q_b + \frac{2c'_a H}{B} + \gamma_1 H^2\left(1 + \frac{2D_f}{H}\right)\frac{K_s \tan \phi'_1}{B} - \gamma_1 H$$

where K_s = punching shear coefficient.

The punching shear coefficient, K_s, is a function of q_2/q_1 and ϕ'_1

Note that q_1 and q_2 are the ultimate bearing capacities of a continuous foundation of width B under vertical load on the surfaces of homogeneous thick beds of upper and lower soil, or

$$q_1 = c'_1 N_{c(1)} + \tfrac{1}{2}\gamma_1 B N_{\gamma(1)}$$

$$q_2 = c'_2 N_{c(2)} + \tfrac{1}{2}\gamma_2 B N_{\gamma(2)}$$

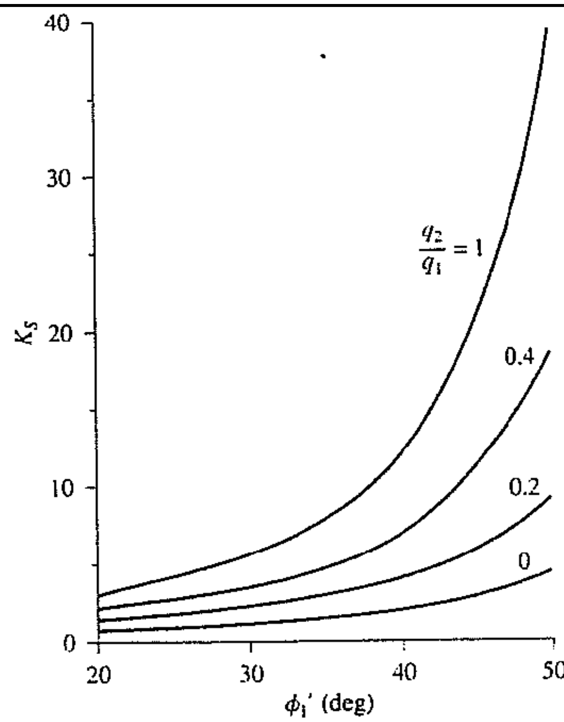


Figure 4.8 Meyerhof and Hanna's punching shear coefficient K_s

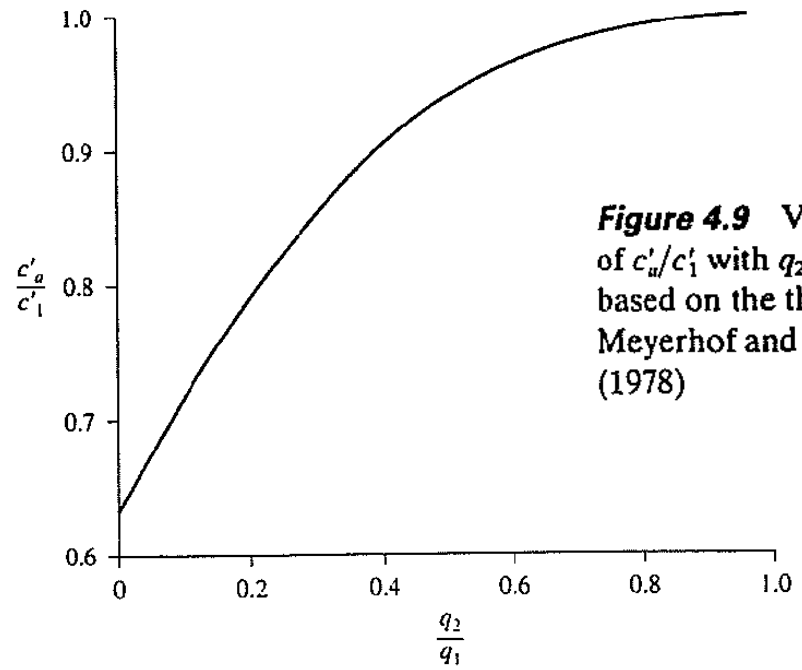
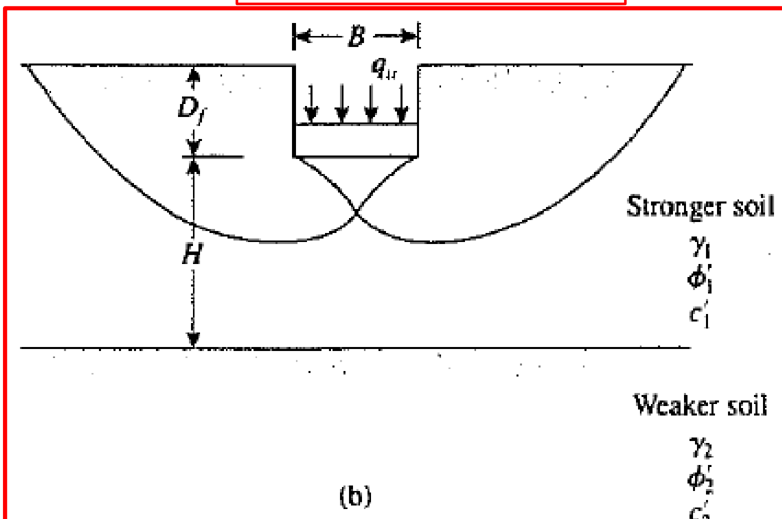


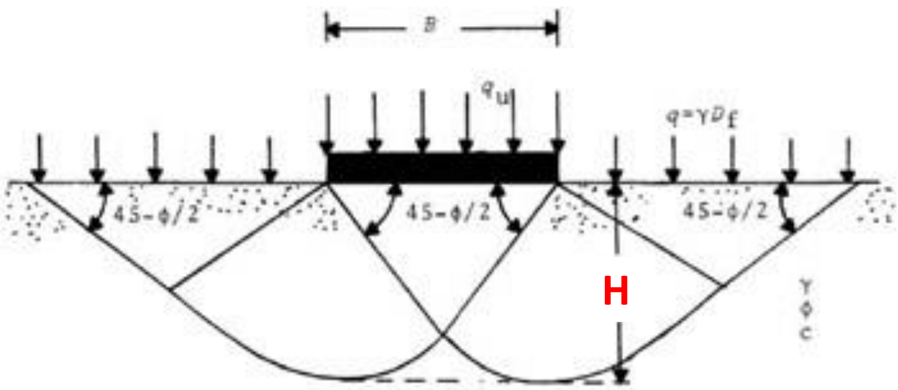
Figure 4.9 Variation of c'_a/c'_1 with q_2/q_1 based on the theory of Meyerhof and Hanna (1978)

However, if the depth H is relatively large, then the failure surface will be completely located in the top soil layer, which is the upper limit for the ultimate bearing capacity.

$$q_u = q_t = \boxed{\frac{1}{2}\gamma \cdot B \cdot N_\gamma + c \cdot N_c + \gamma \cdot D \cdot N_q}$$



How can we check whether the bearing capacity will be controlled only by the upper layer, or both layers?

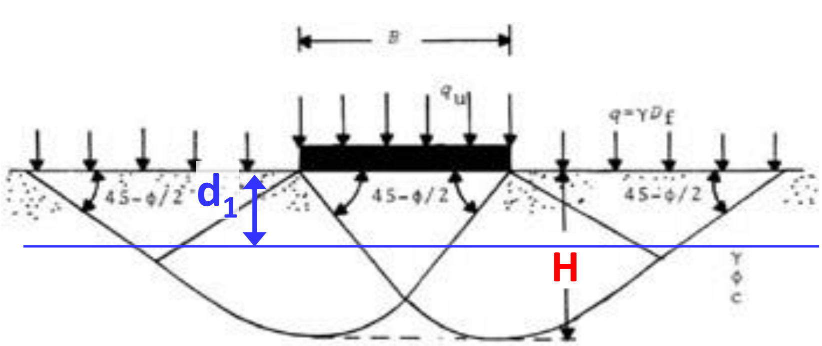


Compute bearing capaciy depth H, $\boxed{H = 0.5B\tan(45+\phi/2)}$
if below foundation upper layer thickness > H....
or < H .....

1. Compute the depth $H = 0.5B\tan(45+\phi/2)$ using ϕ for the top layer.
2. If $H > d_1$ compute the modified value of ϕ for use as[5]

$$\phi' = \frac{d_1\phi_1 + (H - d_1)\phi_2}{H}$$

3. Make a similar computation to obtain c'.
4. Use the bearing-capacity equation for q_{ult} with ϕ' and c'.



Example:

Consider a continuous foundation having width B=2 m, depth of foundation D_f=1.2 m. Soil profile, starting from ground surface, consists of 2.7 m thick sand, underlain by clay.

Top sand layer:

unit weight γ_1=17.5 kN/m^3

ϕ'_1= 40°

c'_1= 0

Bottom clay layer:

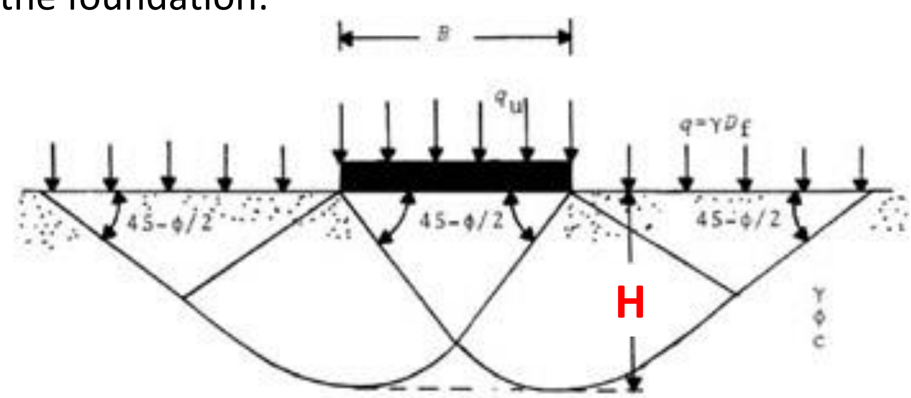
unit weight γ_1=16.5 kN/m^3

ϕ_2= 0°

c_2= 30 kPa

Determine the gross ultimate pressure, q_u

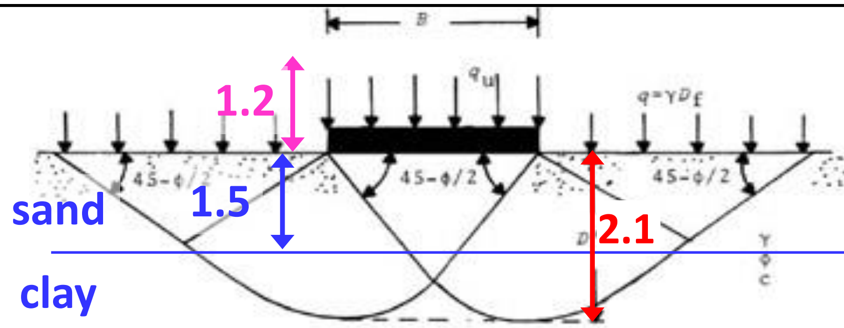
Find bearing capacity influence zone thickness under the foundation:



$$\boxed{H = 0.5B\tan(45 + \phi/2)}$$

H = 0.5 · B · tan(45+ϕ'/2) =

= 0.5 · 2 · tan(45+40/2) = **2.1 m**



Both layers will contribute to bearing capacity

Let's Recall:

$$q_u = q_b + \frac{2c_a'H}{B} + \gamma_1 H^2\left(1 + \frac{2D_f}{H}\right)\frac{K_s \tan\phi_1'}{B} - \gamma_1 H$$

where K_s = punching shear coefficient.

q_b = bearing capacity of the bottom soil layer

The punching shear coefficient, K_s, is a function of q_2/q_1 and ϕ_1'

Note that q_1 and q_2 are the ultimate bearing capacities of a continuous foundation of width B under vertical load on the surfaces of homogeneous thick beds of upper and lower soil, or

$$q_1 = c_1'N_{c(1)} + \tfrac{1}{2}\gamma_1 B N_{\gamma(1)}$$

$$q_2 = c_2'N_{c(2)} + \tfrac{1}{2}\gamma_2 B N_{\gamma(2)}$$

$$q_u = q_b + \frac{2 \cdot c'_a \cdot H}{B} + \gamma_1 \cdot H^2 \left(1 + \frac{2 \cdot D_f}{H}\right) \frac{K_s \cdot tan\emptyset'_1}{B} - \gamma_1 \cdot H$$

$$q_1 = \frac{1}{2}\gamma B N_\gamma + cN_c$$

$$q_1 = \frac{1}{2} \cdot 17.5 \cdot 2 \cdot 100 = 1750\ kPa$$

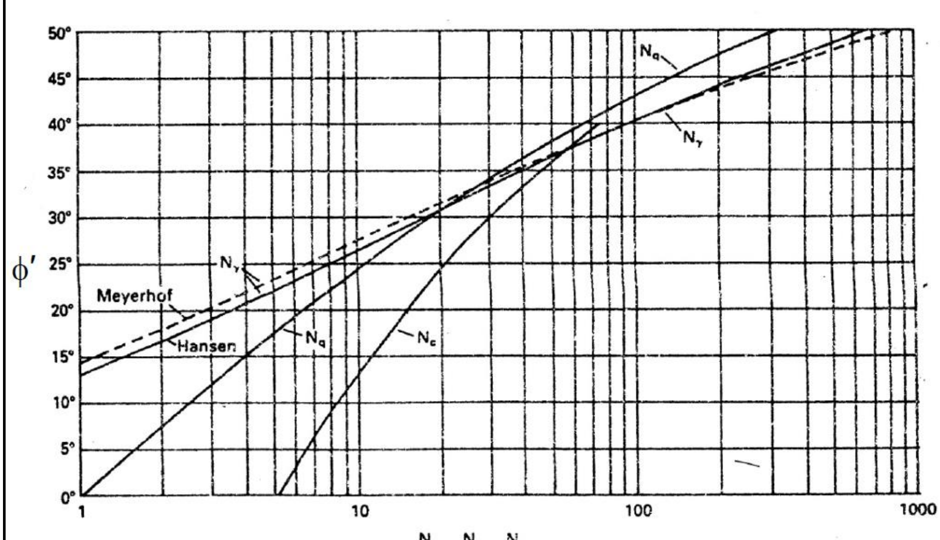
$$q_2 = c_u N_c = 30 \cdot 6 = 180\ kPa$$

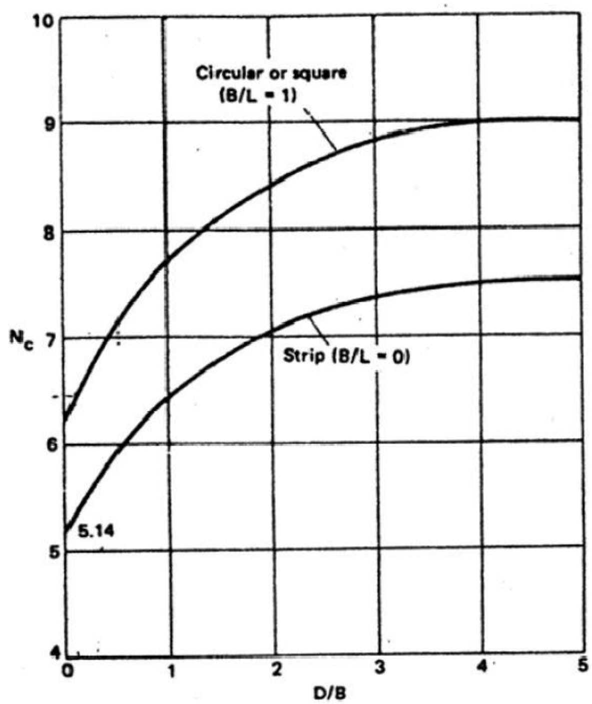
$$\frac{q_2}{q_1} = \frac{180}{1750} = 0.1 \qquad K_s \approx 3$$

$$q_b = c_u N_c + \gamma D N_q = 30 \cdot 6.7 + 16.5 \cdot 2.7 \cdot 1 = 246\ kPa$$

$$q_u = 246 + \frac{2 \cdot 0 \cdot 1.5}{2} + 17.5 \cdot 1.5^2 \left(1 + \frac{2 \cdot 1.2}{1.5}\right) \frac{3 \cdot tan40}{2} - 17.5 \cdot 1.5$$

$$q_u = 246 + 0 + 129 - 26 = 349\ kPa$$





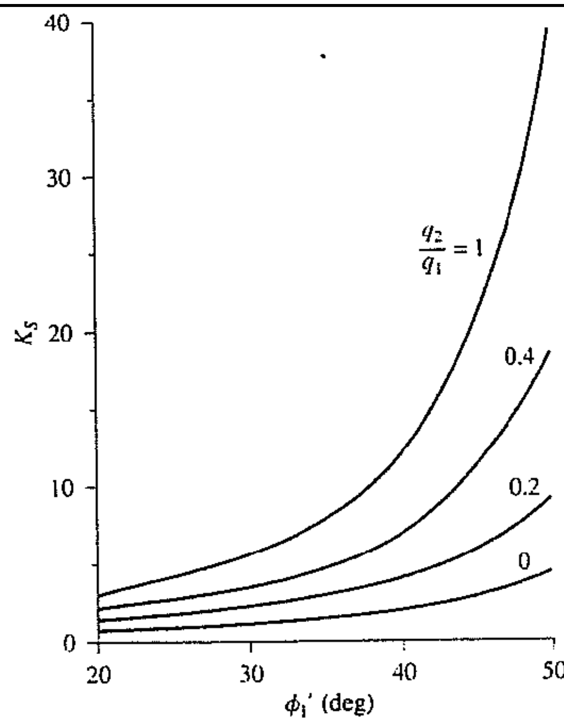


Figure 4.8 Meyerhof and Hanna's punching shear coefficient K_s

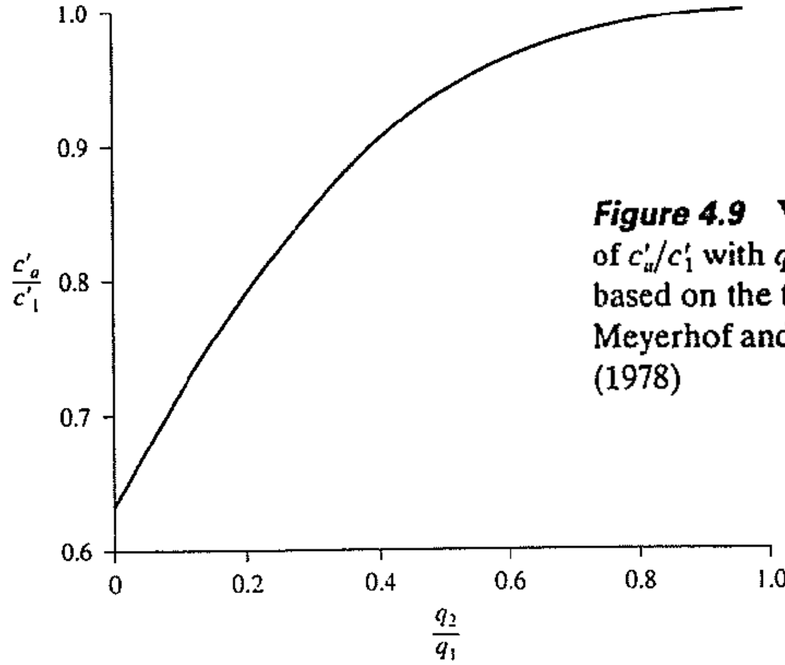
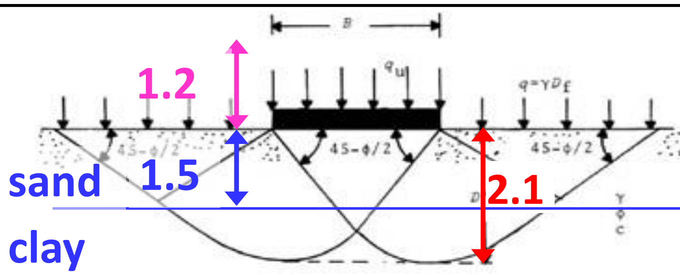


Figure 4.9 Variation of c'_a/c'_1 with q_2/q_1 based on the theory of Meyerhof and Hanna (1978)



Can we use an Alternative method?: Use weighted average soil properties?

$$\emptyset' = \frac{1.5 \times 40 + 0.6 \times 0}{2.1} = 28.6°$$

$$\phi' = \frac{d_1\phi_1 + (H - d_1)\phi_2}{H}$$

$$c' = \frac{1.5 \times 0 + 0.6 \times 30}{2.1} = 8.6\ kPa$$

$$\gamma = \frac{1.5 \times 17.5 + 0.6 \times 16.5}{2.1} = 17.2\ kN/m^3$$

$$q_u = \frac{1}{2}\gamma B N_\gamma + cN_c + \gamma D N_q$$

$$q_u = \frac{1}{2} \cdot 17.2 \cdot 2 \cdot N_\gamma + 8.6 \cdot N_c + 17.5 \cdot 1.2 \cdot N_q$$

$$q_u = \frac{1}{2} \cdot 17.2 \cdot 2 \cdot 16 + 8.6 \cdot 29 + 17.5 \cdot 1.2 \cdot 17$$

$$q_u = 275 + 249 + 357 = 881\ kPa$$

\`Using weighted average soil properties\` method does not consider individual contributions in each term in bearing capacity equation (triangular wedge zone, spiral failure zone etc), by upper soil and lower soil separately. This is a very rough method?!

Dynamic compaction or Heavy Tamping