



جزوه باما

دانلود جزوات، نمونه سؤالات
و پروپوزنت‌های دانشگاهی

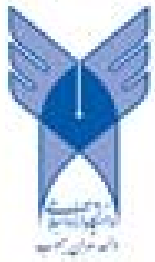
Jozvebama.ir



ظرفیت باربری شالوده های سطحی

Bearing Capacity

of shallow foundation systems

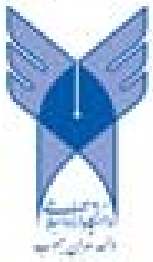


واژگان

پی : مجموعه بخش هایی از سازه و خاک در تماس با آن که وظیفه انتقال بار بین سازه و زمین از طریق آن صورت می پذیرد.

شالوده : پی هایی که در عمق کم و نزدیک سطح زمین اجرا می شوند.

شمع : پی هایی که نسبت عمق به کوچکترین بعد افقی آنها از ۶ بزرگتر باشد.



DEFINITIONS OF KEY TERMS

Foundation is a structure that transmits loads to the underlying soils.

Footing is a foundation consisting of a small slab for transmitting the structural load to the underlying soil. Footings can be individual slabs supporting single columns (Figure a) or combined to support two or more columns (Figure b), or be a long strip of concrete slab (Figure 12.2c, d; width B to length L ratio is small, i.e., it approaches zero) supporting a load-bearing wall, or a mat (Figure e).

Embedment depth (D_f) is the depth below the ground surface where the base of the foundation rests.

Shallow foundation is one in which the ratio of the embedment depth to the minimum plan dimension, which is usually the width (B), is $D_f/B < 2.5$.

Ultimate bearing capacity is the maximum pressure that the soil can support.

Ultimate net bearing capacity (q_u) is the maximum pressure that the soil can support above its current overburden pressure.

Ultimate gross bearing capacity (q_{ult}) is the sum of the ultimate net bearing capacity and the overburden pressure above the footing base.

Allowable bearing capacity or **safe bearing capacity (q_a)** is the working pressure that would ensure a margin of safety against collapse of the structure from shear failure. The allowable bearing capacity is usually a fraction of the ultimate net bearing capacity.

Factor of safety or **safety factor (FS)** is the ratio of the ultimate net bearing capacity to the allowable net bearing capacity or to the applied maximum net vertical stress. In geotechnical engineering, a factor of safety between 2 and 5 is used to calculate the allowable bearing capacity.

Ultimate limit state defines a limiting stress or force that should not be exceeded by any conceivable or anticipated loading during the design life of a foundation or any geotechnical system.

Serviceability limit state defines a limiting deformation or settlement of a foundation, which, if exceeded, will impair the function of the structure that it supports



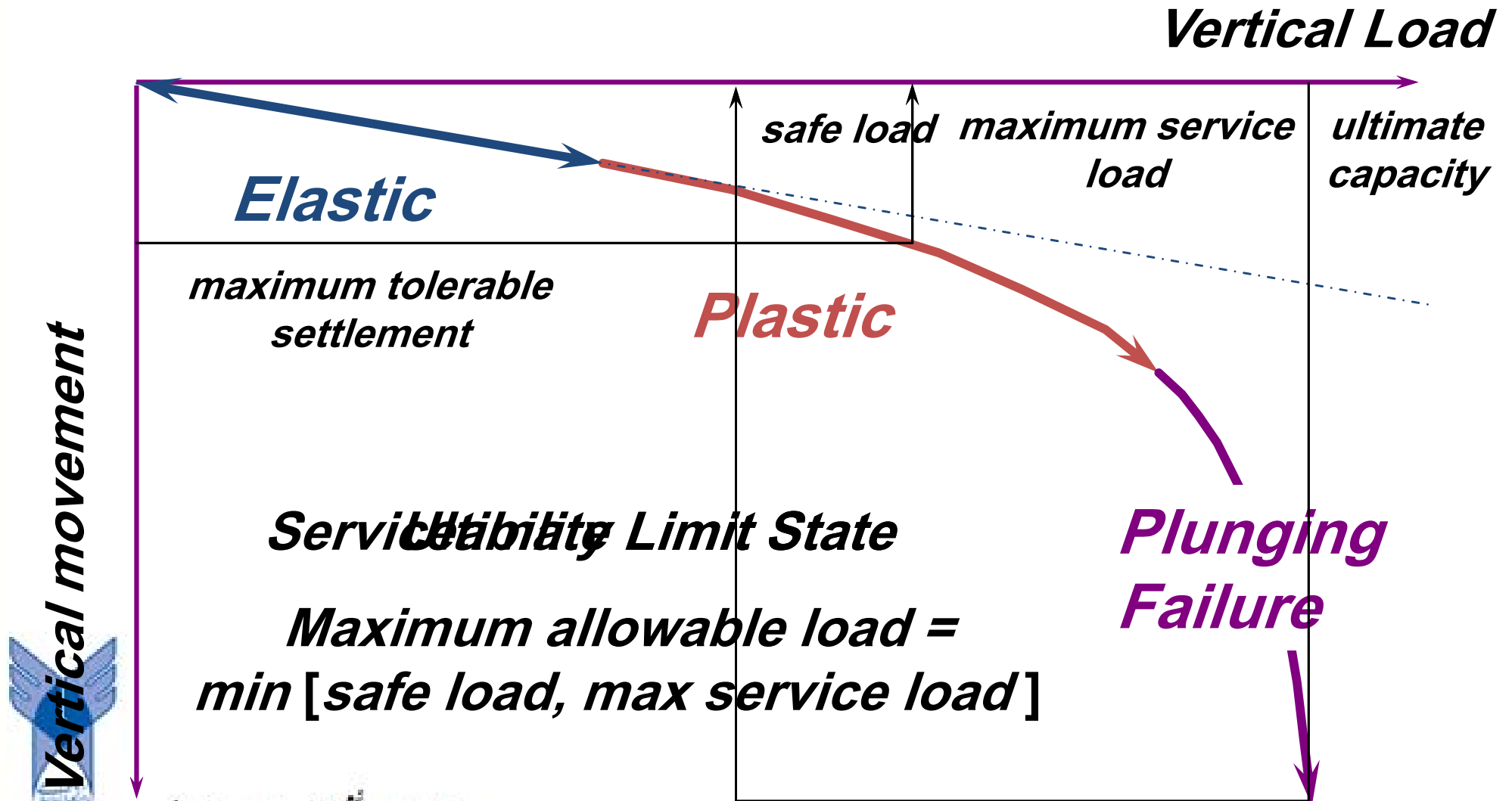
ظرفیت باربری مجاز

1. Safe bearing capacity or *strength*
2. Allowable bearing pressure

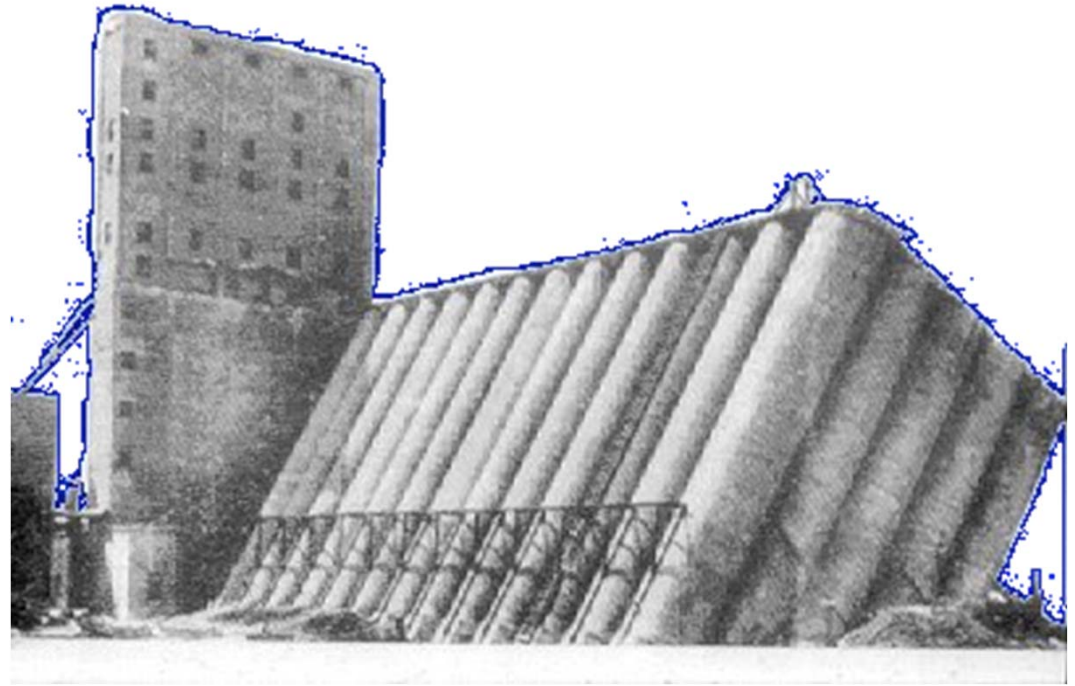
ULTIMATE bearing capacity	q_{ult}	حداکثر تنش قابل تحمل برای پی
SAFE bearing capacity	q_{safe}	پس از اعمال ضریب اطمینان بر q_{ult}
SERVICE bearing pressure	q_{serv}	حداکثر بار بر اساس معیار نشست
ALLOWABLE bearing capacity	q_{all}	حداکثر بار مجاز قابل اعمال بر پی



رفتار شالوده



حالات حدی



Serviceability

Ultimate

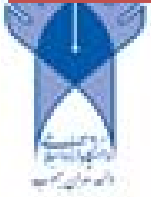
گسیختگی

Rotational Failure

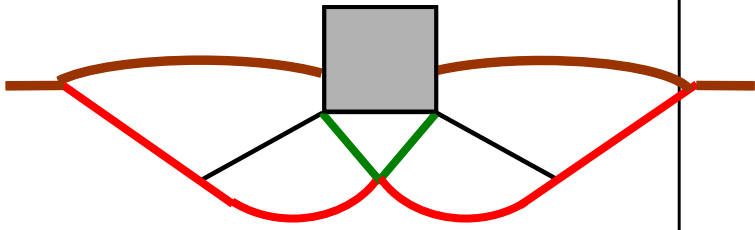
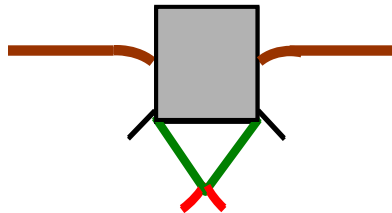
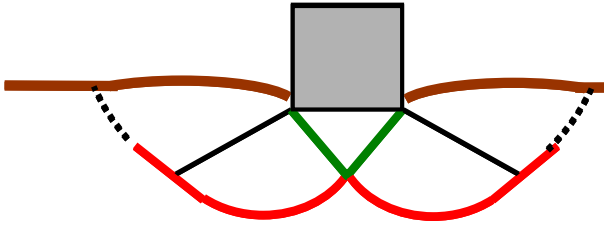
Force

Resistance

Soil Heave

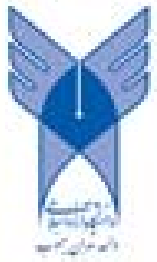
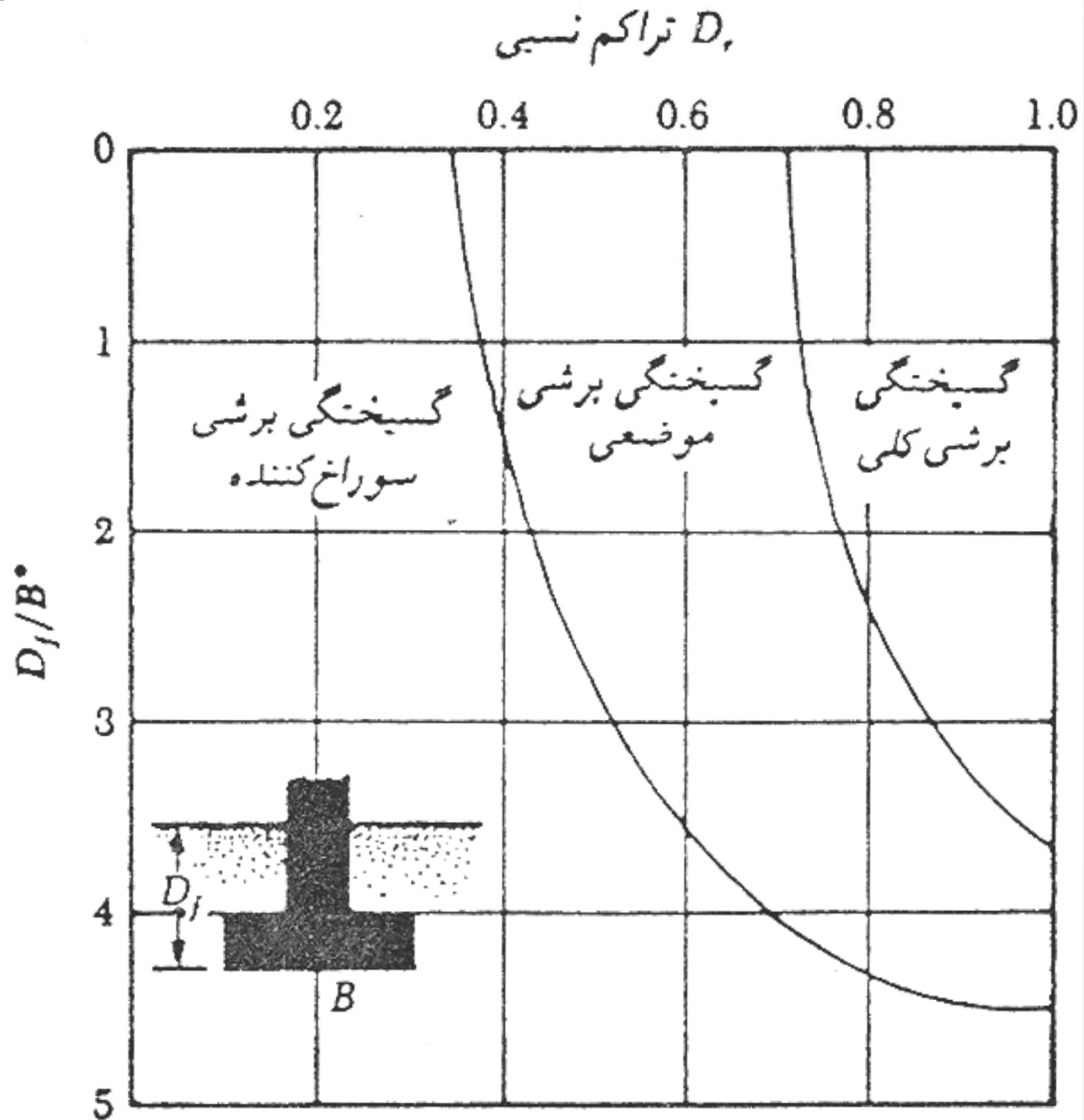


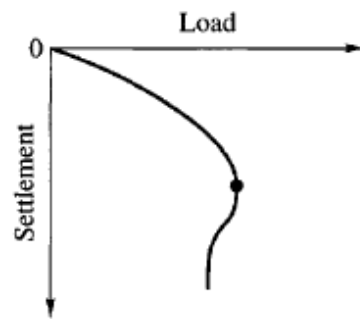
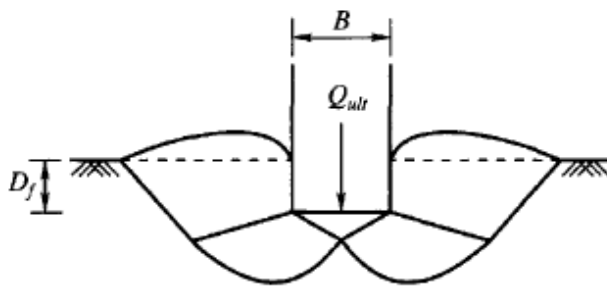
مدل های مختلف گسیختگی

<p><i>General shear</i></p> <p>گسیختگی برشی کلی</p>	<p><i>Punching shear</i></p> <p>گسیختگی برشی پانچ</p>	<p><i>Local shear</i></p> <p>گسیختگی برشی موضعی</p>
		
<p>Fairly incompressible soil</p>	<p>Soft, <u>very</u> <u>compressible</u> soil</p>	<p>Intermediate soil condition</p>

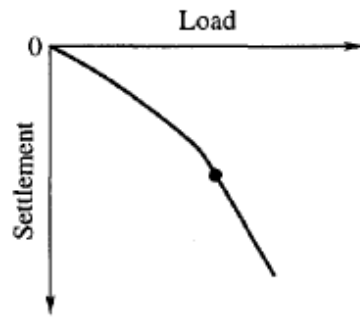
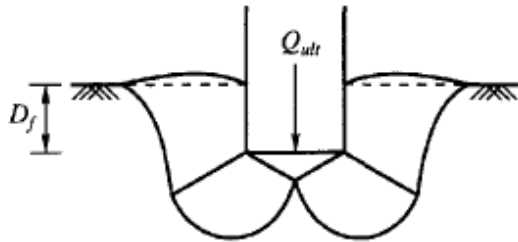


معیار مدل گسیختگی

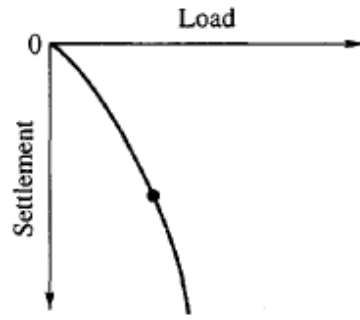
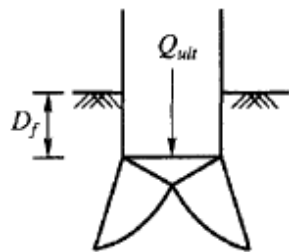




(a) General shear failure



(b) Local shear failure



(c) Punching shear failure

Figure 12.4 Modes of bearing capacity failure (Vesic, 1963)

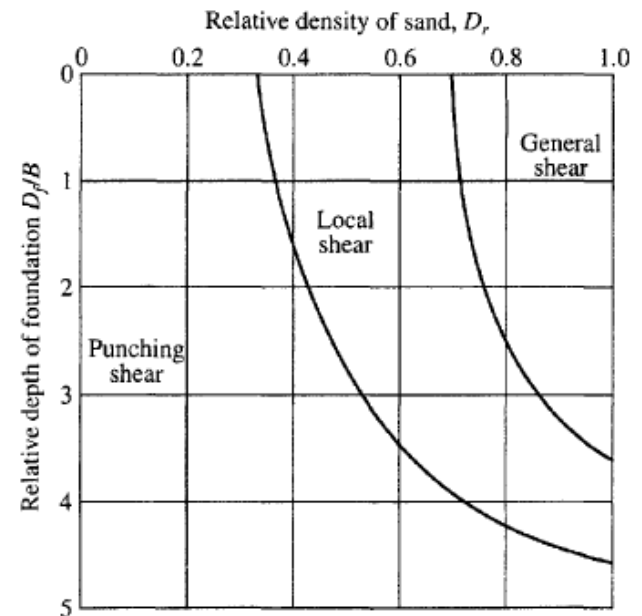
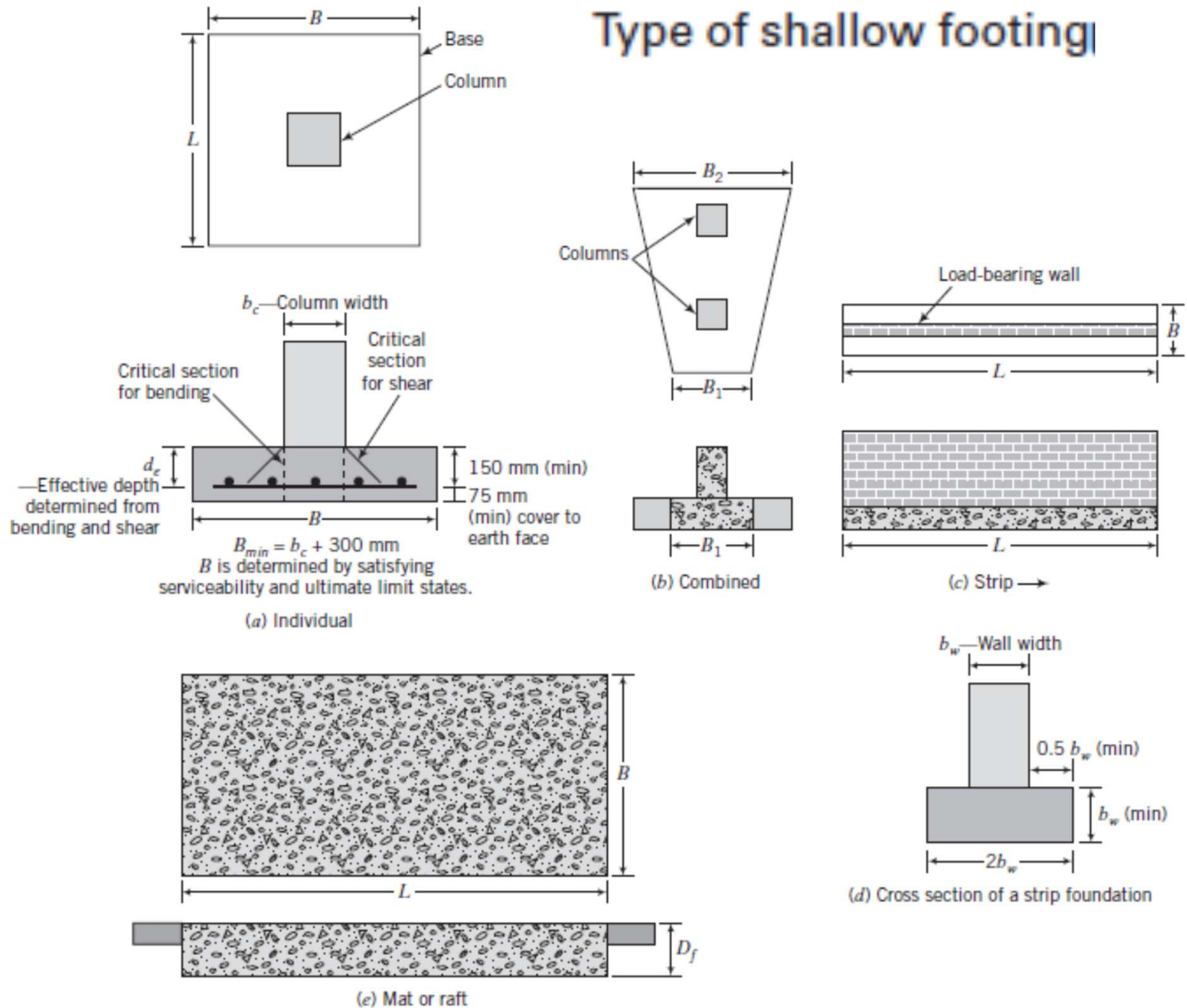


Figure 12.5 Modes of failure of model footings in sand (after Vesic, 1963)

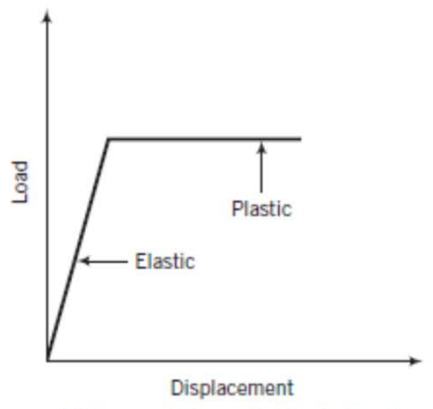


BEARING CAPACITY OF SOILS

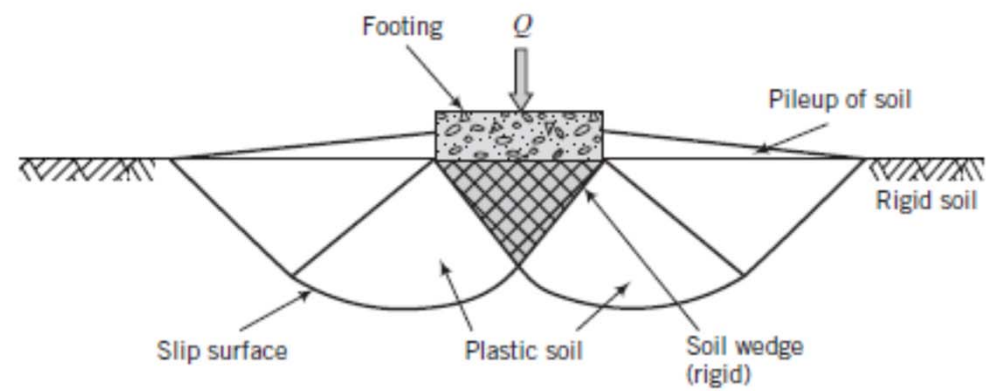
Type of shallow footing



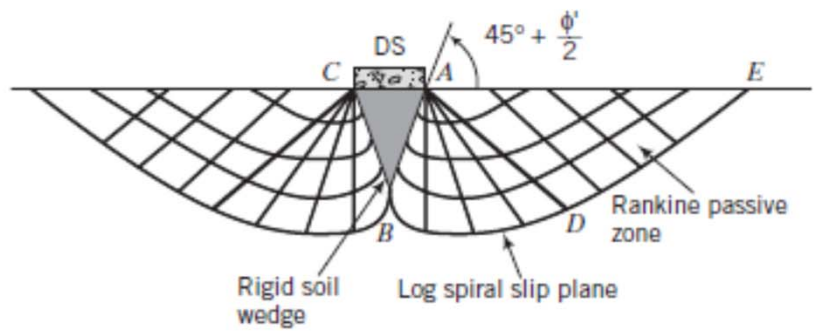
Soil Response to a Loaded Footing



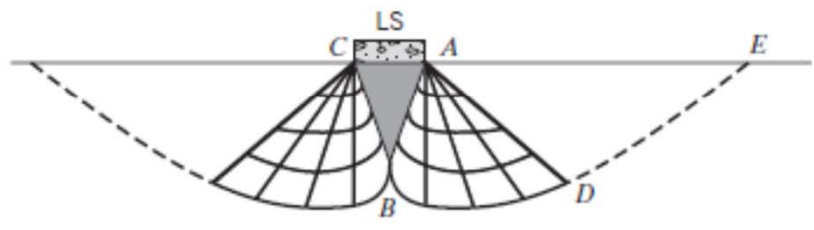
(a) Linear elastic-perfectly plastic soil



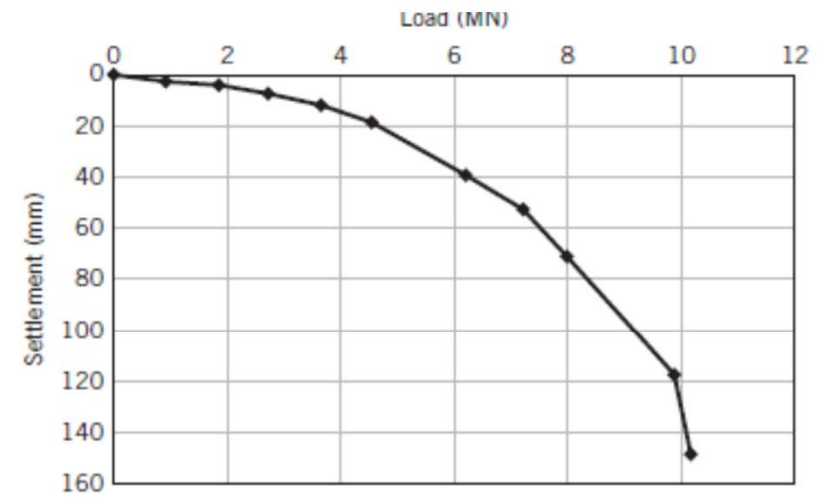
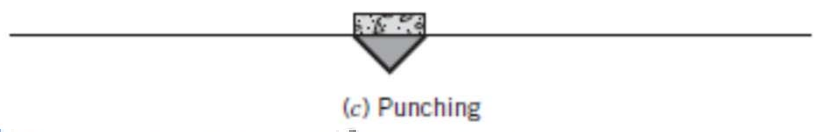
(b) Failure mechanism



(a) General shear failure



(b) Local shear failure



(c) Shallow footing test results



Conventional failure mechanisms

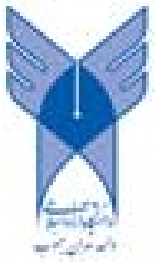
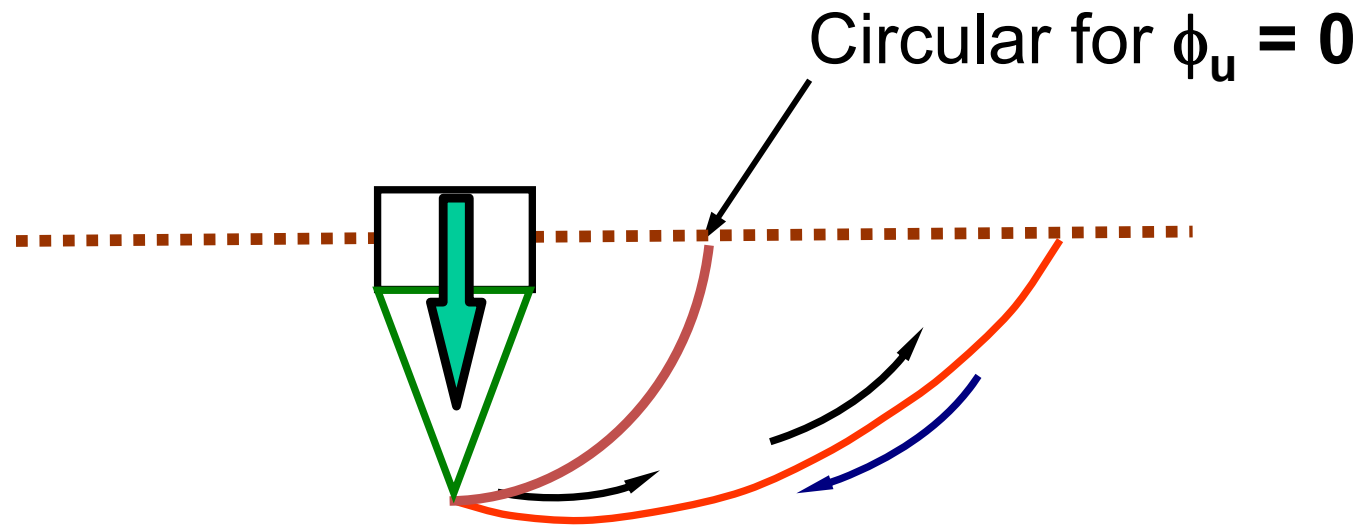
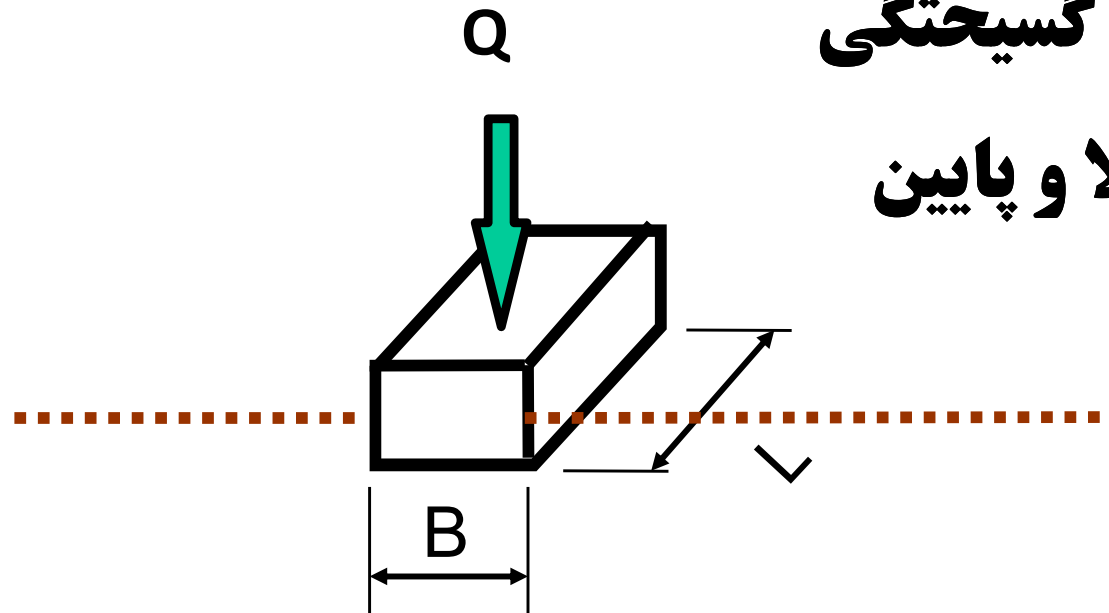
روش های تعیین ظرفیت باربری نهایی

- آزمایش بار گذاری تمام مقیاس
- آزمایش بار گذاری بر روی مدل های مقیاس شده
- آنالیز تعادل حدی
- روش های عددی مثل اجزاء محدود



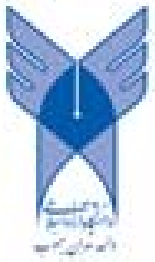
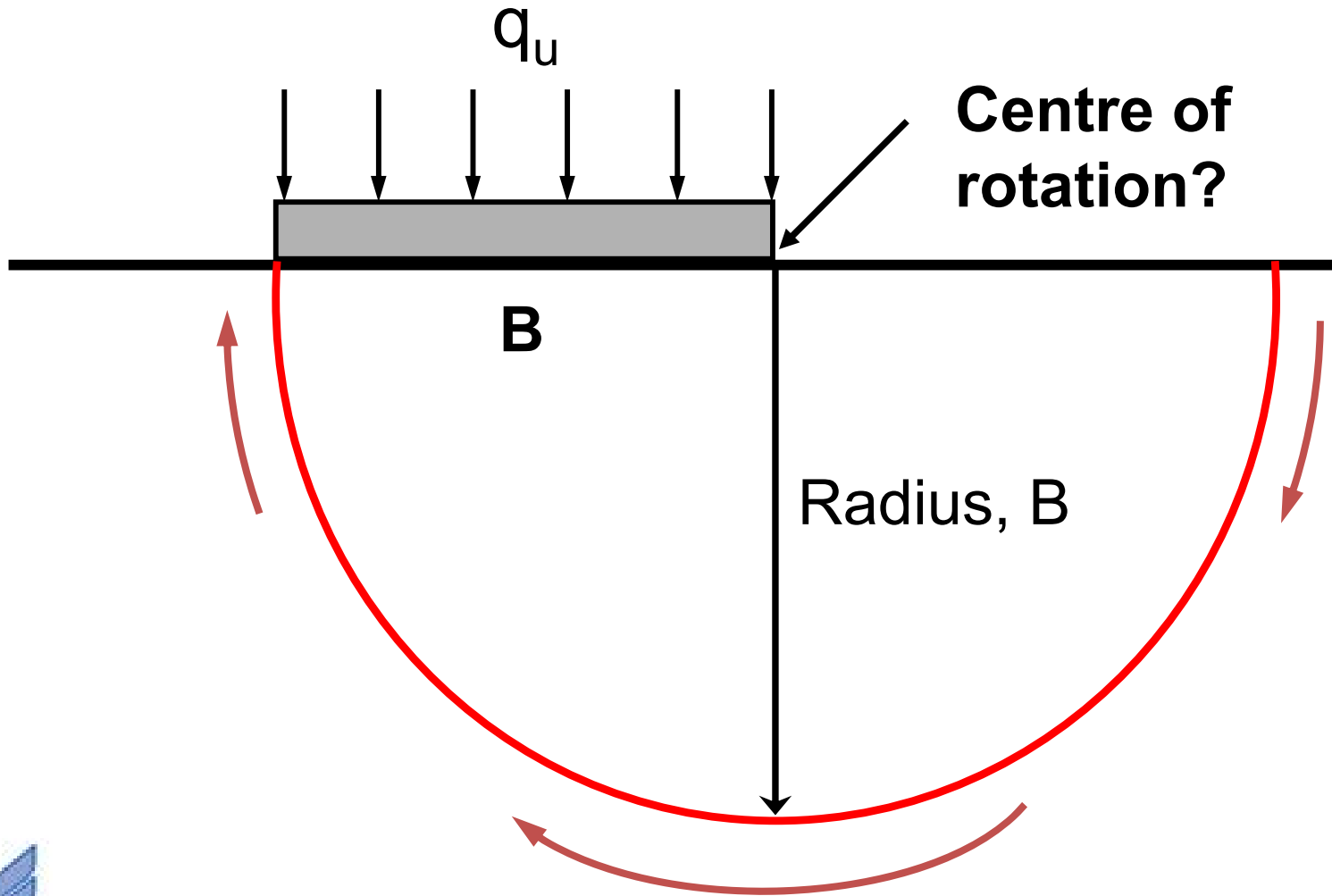
مکانیسم گسیختگی

حد بالا و پایین



روش ساده حد بالا

for $\phi_u = 0$ soils



حل

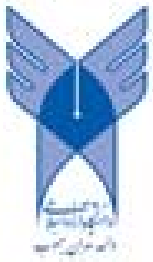
با ممان گیری حول نقطه مرکز (گوشه پی) داریم :

Disturbing moment = restoring moment

$$q_{ull} B(0.5B) = (c_u \pi B) B$$

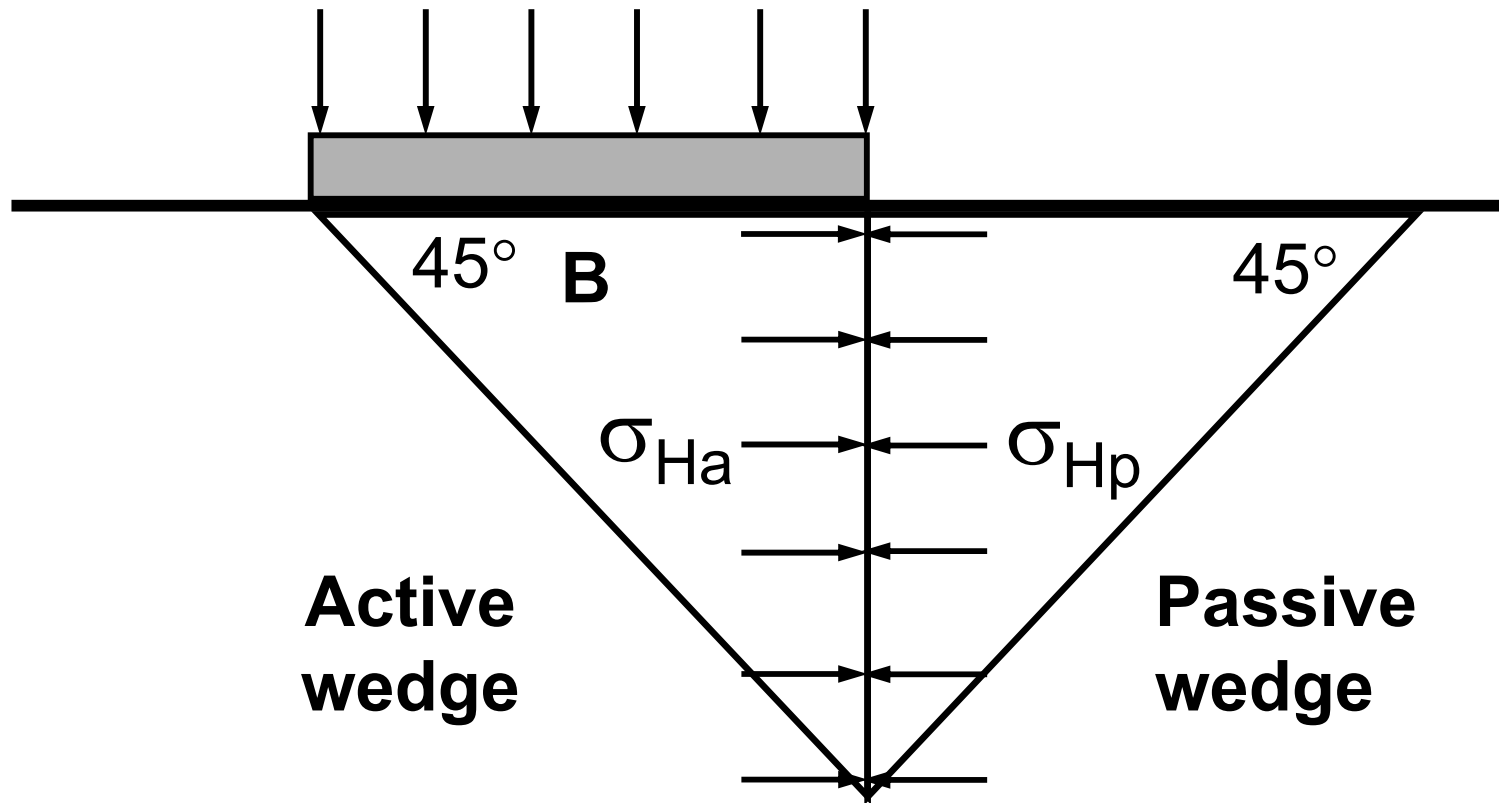
$$\therefore q_{ull} = (2\pi) c_u$$

$$q_{ull} = 6.3 c_u$$

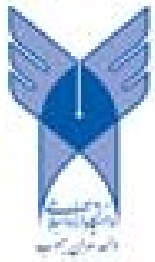


روش ساده حد پایین

for $\phi_u = 0$ soils

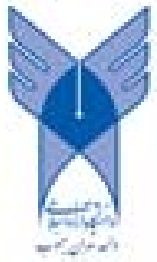
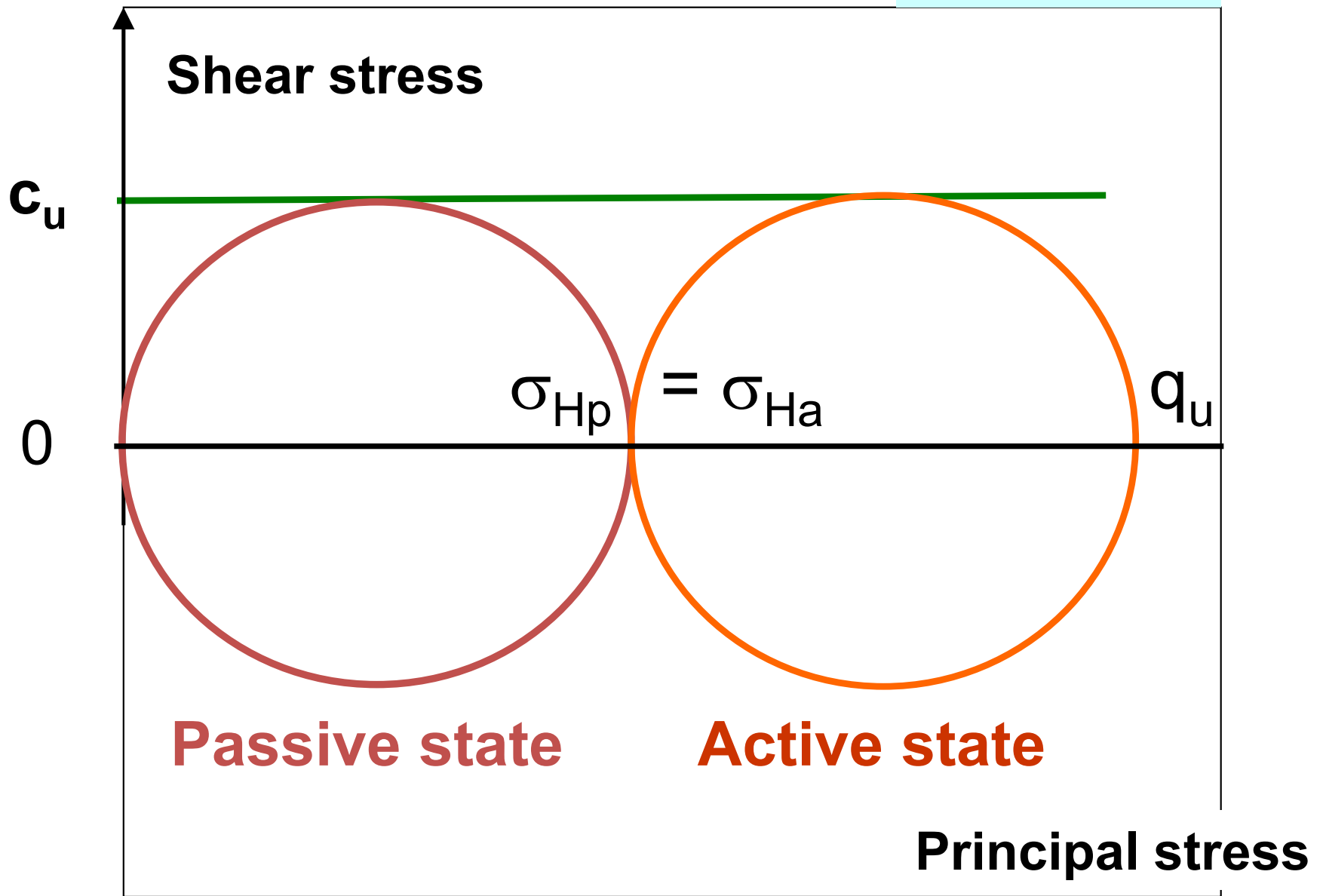


$$K_a = K_p = 1$$



حل

$$q_u = 4c_u$$



جواب دقیق

- Lower bound estimate (safe)

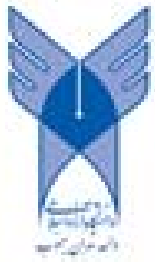
- $q_u = 4c_u$

The correct answer:

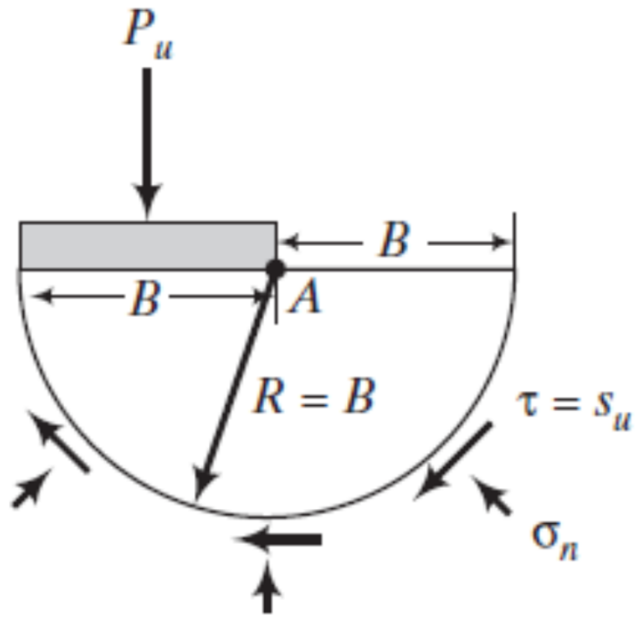
$$q_u = 5.14c_u$$

- Upper bound estimate (unsafe)

- $q_u = 6.3c_u$

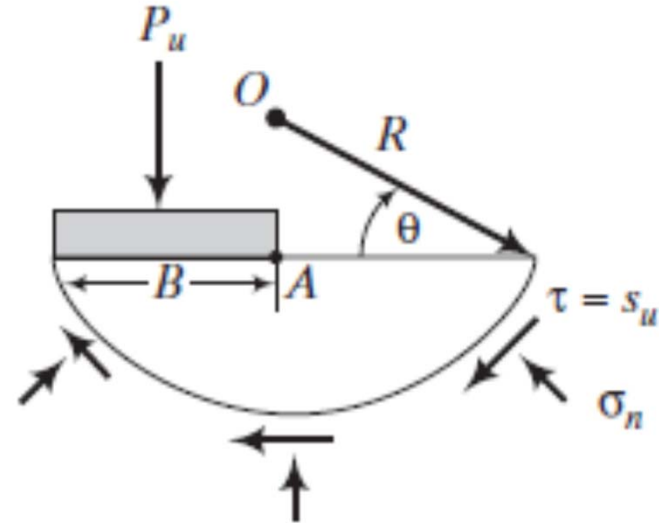


COLLAPSE LOAD USING THE LIMIT EQUILIBRIUM METHOD



$$P_u \times \frac{B}{2} - s_u \pi B \times B = 0$$

$$P_u = 6.28 B s_u$$



$$P_u(R \cos \theta - B/2) - s_u[(\pi - 2\theta)R]R = 0$$

$$P_u = \frac{s_u[(\pi - 2\theta)R]R}{(R \cos \theta - B/2)} = \frac{s_u(\pi - 2)R}{(\cos \theta - B/2R)}$$

$$\frac{\partial P_u}{\partial R} = \frac{4s_u R(\pi - 2\theta)(R \cos \theta - B)}{(2R \cos \theta - B)^2} = 0$$

$$\frac{\partial P_u}{\partial \theta} = \frac{4s_u R^2(B - 2R \cos \theta + \pi R \sin \theta - 2R\theta \sin \theta)}{(2R \cos \theta - B)^2} = 0$$

$$\theta = 23.2^\circ \text{ and } R = B \sec \theta \Rightarrow P_u = 5.52 B s_u$$

The exact solution to our problem, using more complex analysis than the limit equilibrium method

$$P_u = 5.14 B s_u$$

روش تحلیل ایستایی - دانشگاه ارومیه اسلامی واحد عمران آذربایجان



مکانیسم ترزاقی برای گسیختگی

۱. مکانیسم گسیختگی خاک مشابه شکل (۱۰-۱۷) است. به عبارت دیگر خاک در حالت گسیختگی برشی کلی گسیخته می شود.
 ۲. خطوط گسیختگی تا سطح اک امتداد نمی یابند.
 ۳. زاویه خطوط گسیختگی گوه با افق برابر ϕ' فرض شده است.
 ۴. عمق پی کمتر از عرض آن است، یعنی $D \leq B$.
 ۵. از اثر خاک بالای تراز کف پی صرف نظر شده و فقط وزن آن ($\bar{q} = \gamma D$) در نظر گرفته می شود.
 ۶. کف پی زیر است. بنابراین احتمال لغزش فصل مشترک خاک و پی منفی است.
- ناحیه II از یک کمان دایره و ناحیه III از یک خط راست تشکیل شده است.

۱- گوه زیر پی

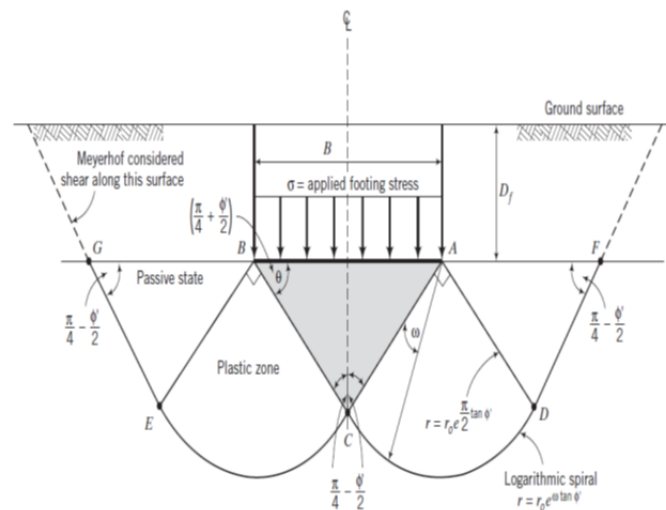
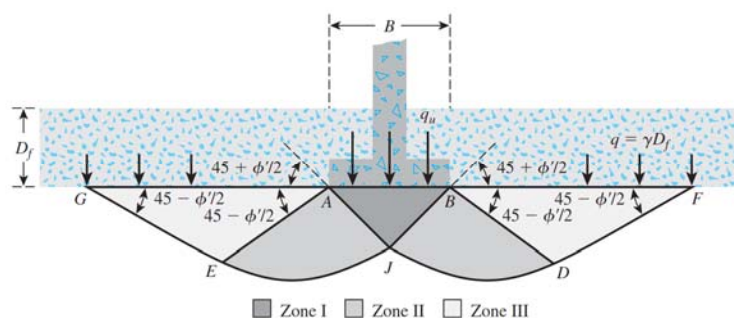
• زاویه گوه ϕ'
 ۲- ناحیه پلاستیک (برش شعاعی)

• کمان دایره

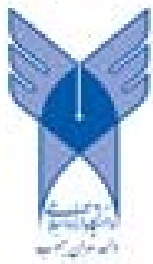
۳- ناحیه پاسیو

• زاویه گوه $(45^\circ - \phi'/2)$

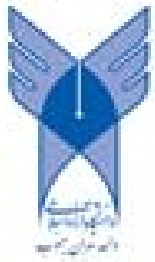
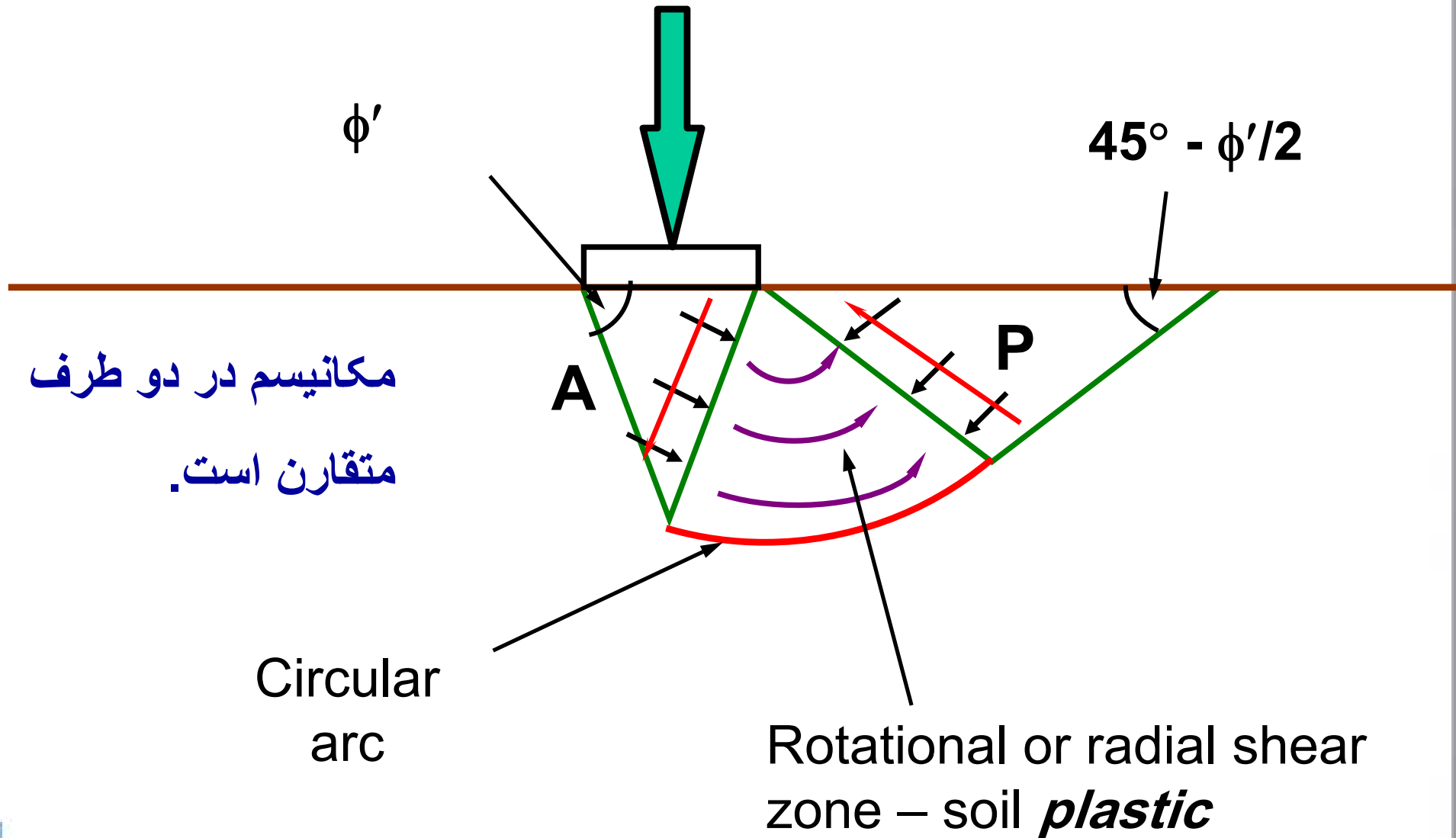
• خط مستقیم



شکل ۱۰-۱۷ مکانیسم گسیختگی ترزاقی



مکانیسم ترزاقی برای گسیختگی



رابطه عمومی ترزاقی

نکته: ساده ترین شکل معادله ظرفیت باربری یک پی نواری به صورت زیر است:

$$q_u = CN_c + \bar{q}N_q + 0.5\gamma BN_\gamma \quad (10-1)$$

که در آن

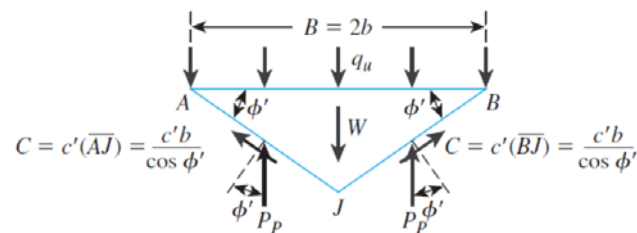
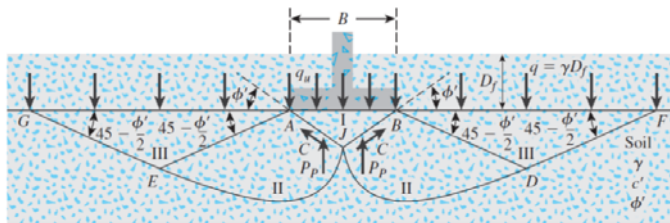
q_u = ظرفیت باربری نهایی پی

C = ضریب چسبندگی

\bar{q} = تنش های موثر وارد بر تراز کف پی ($\bar{q} = \gamma D$)

B = عرض پی نواری

N_c, N_q, N_γ = ضرایب ظرفیت باربری



رابطه عمومی ترزاقی

N_q, N_c, N_γ = ضریب ظرفیت باربری ترزاقی که مقدار آنها را می توان از جدول (۱۰-۱) یا روابط زیر محاسبه کرد:

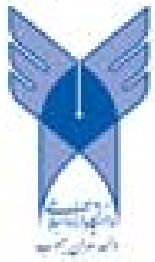
$$N_q = \frac{a^2}{2 \cos^2(45 + \varphi'/2)}; \quad a = e^{(0.75 \pi - \varphi'/2) \tan \varphi'}$$

$$N_c = (N_q - 1) \cot \varphi'$$

$$N_\gamma = \frac{\tan \varphi'}{2} \left(\frac{k_{p\gamma}}{\cos^2 \varphi'} - 1 \right)$$

S_c و S_γ = ضرایب شکل که بسته به نوع پی عبارتند از :

	پی نواری	پی دایره ای	پی مربعی
S_c	۱.۰	۱.۳	۱.۳
S_γ	۱.۰	۰.۶	۰.۸



Terzaghi Bearing Capacity Equation

- *for a long “strip” footing*

Cohesion term

↓

Soil self-weight term

↓

$$q_u = (c'N_c) + (q_o N_q) + (0.5\gamma BN_\gamma)$$

↑

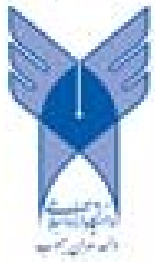
Surface surcharge term



جدول ۱-۱ ضرایب ظرفیت باربری ترزاقی

ϕ' (deg)	N_c	N_q	N_γ^a	ϕ' (deg)	N_c	N_q	N_γ^a
0	5.70	1.00	0.00	26	27.09	14.21	9.84
1	6.00	1.10	0.01	27	29.24	16.90	11.60
2	6.30	1.22	0.04	28	31.61	17.81	13.70
3	6.62	1.35	0.06	29	34.24	19.98	16.18
4	6.97	1.49	0.10	30	37.16	22.46	19.13
5	7.34	1.64	0.14	31	40.41	25.28	22.65
6	7.73	1.81	0.20	32	44.04	28.52	26.87
7	8.15	2.00	0.27	33	48.09	32.23	31.94
8	8.60	2.21	0.35	34	52.64	36.50	38.04
9	9.09	2.44	0.44	35	57.75	41.44	45.41
10	9.61	2.69	0.56	36	63.53	47.16	54.36
11	10.16	2.98	0.69	37	70.01	53.80	65.27
12	10.76	3.29	0.85	38	77.50	61.55	78.61
13	11.41	3.63	1.04	39	85.97	70.61	95.03
14	12.11	4.02	1.26	40	95.66	81.27	116.31
16	12.86	4.45	1.52	41	106.81	93.85	140.51
16	13.68	4.92	1.82	42	119.67	108.75	171.99
17	14.60	5.45	2.18	43	134.58	126.50	211.56
18	15.12	6.04	2.59	44	161.95	147.74	261.60
19	16.56	6.70	3.07	45	172.28	173.28	325.34
20	17.69	7.44	3.64	46	196.22	204.19	407.11
21	18.92	8.26	4.31	47	224.55	241.80	512.84
22	20.27	9.19	5.09	48	258.28	287.85	650.67
23	21.75	10.23	6.00	49	298.71	344.63	831.99
24	23.36	11.40	7.08	50	347.50	416.14	1072.80
25	25.13	12.72	8.34				

^a N_γ values from Kumbhojkar (1993)



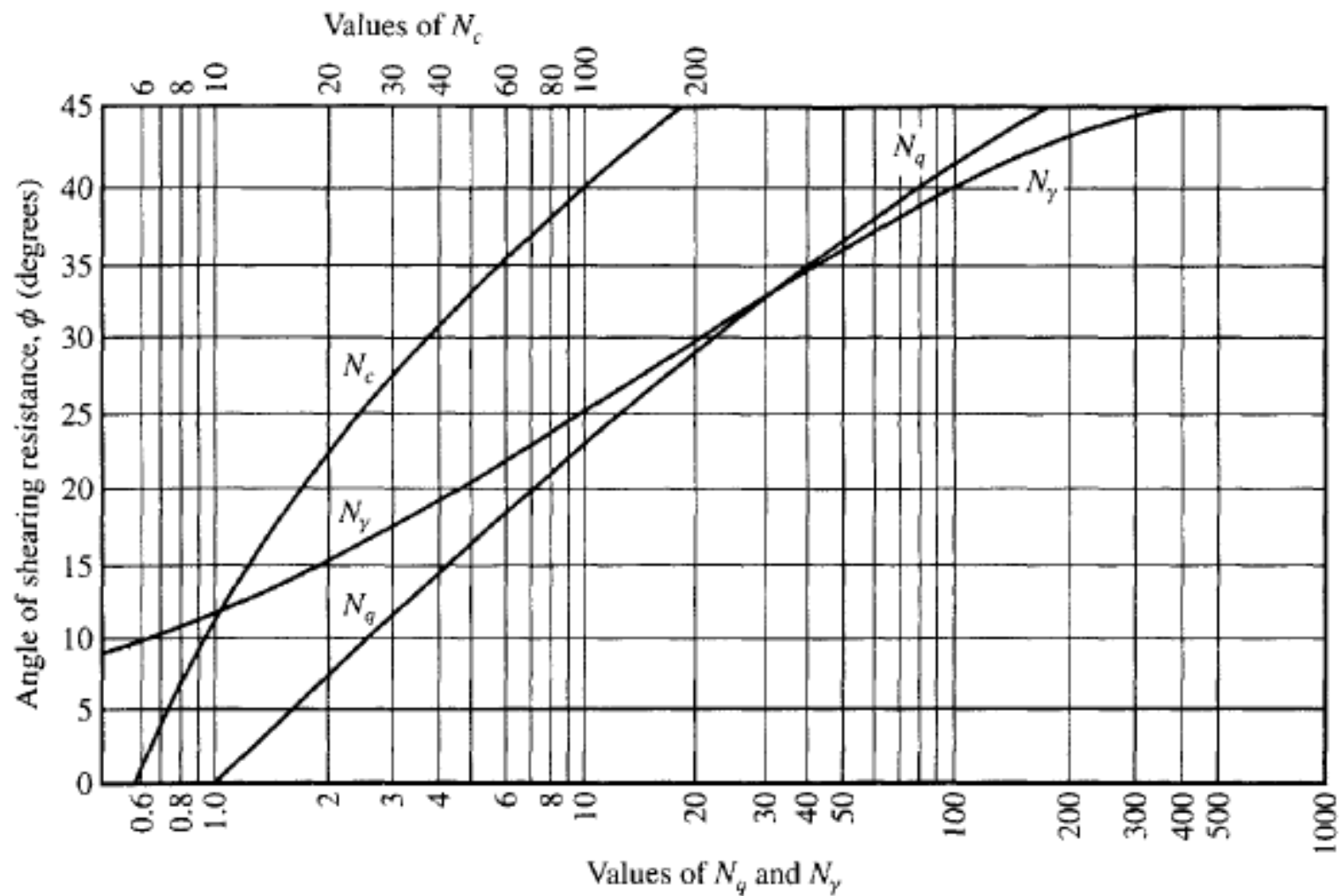


Figure 12.7 Terzaghi's bearing capacity factors for general shear failure



(a) Total Overburden Pressure q_o

q_o is the intensity of total overburden pressure due to the weight of both soil and water at the base level of the foundation.

$$q_o = \gamma D_{w1} + \gamma_{\text{sat}} \bar{D}_w \quad (12.1)$$

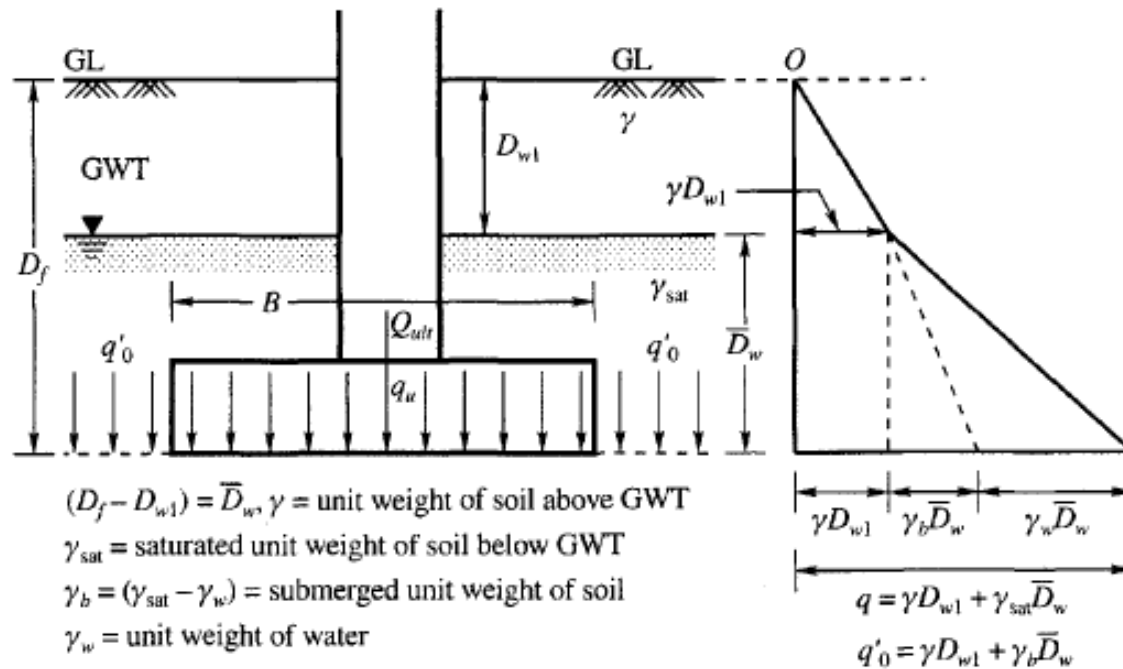


Figure 12.3 Total and effective overburden pressures

(b) Effective Overburden Pressure q'_0

q'_0 is the effective overburden pressure at the base level of the foundation.

$$q'_0 = \gamma D_{w1} + \gamma_b \bar{D}_w \quad (12.2)$$

when $\bar{D}_w = 0$, $q'_0 = \gamma D_{w1} = \gamma D_f$.



نکته: برای محاسبه ظرفیت باربری مجاز با استفاده از q_{ult} از یکی از روابط زیر استفاده می شود:

$$q_{all} = \frac{q_{ult}}{S.F.} \quad (الف-۱۰-۳)$$

$$q_{all(net)} = \frac{q_{ult} - \bar{q}}{S.F.} \quad (ب-۱۰-۳)$$

$$q_{all(net)} = \frac{q_{ult}}{S.F.} - \bar{q} \quad (ج-۱۰-۳)$$

که در آن

q_{all} = ظرفیت باربری مجاز (allowable)

$q_{all(net)}$ = ظرفیت باربری مجاز خالص

q_{ult} = ظرفیت باربری نهایی

\bar{q} = تنش موثر وارد بر تراز کف پی ($\bar{q} = \gamma D$)

$S.F.$ = ضریب اطمینان (safety factor)

در تذکره‌های زیر فلسفه هر یک از روابط فوق توضیح داده خواهد شد.

تذکر (۱): در رابطه (الف-۱۰-۳) مقدار q_{ult} بر یک ضریب اطمینان بزرگتر از واحد تقسیم می شود

به این ترتیبی مقدار q_{all} به اندازه کافی کمتر از q_{ult} خواهد بود.

تذکر (۲): مهندسانی که از رابطه (ب-۱۰-۳) استفاده میکنند، معتقدند که باید باری که به تراز کف

پی وارد می شود از ظرفیت باربری نهایی آن کم شود ($q_{ult} - \bar{q}$) و سپس این مقدار بر ضریب اطمینان

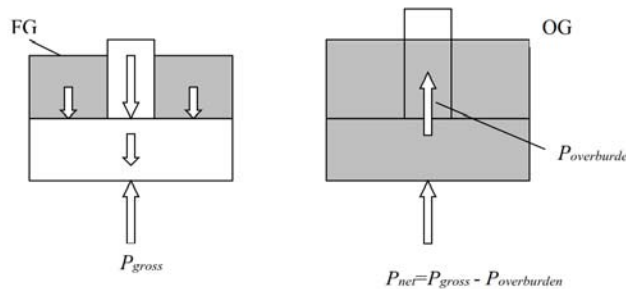
تقسیم شود. با کمی تامل می توان دریافت که برای این کار یک دلیل کاملاً منطقی وجود دارد. اگر فرض

کنیم که خاک زیر پی قادر به تحمل تنشی معادل q_{ult} باشد و پی در عمق D واقع شده باشد، واضح

است که در این عمق تنشی معادل $\bar{q} = \gamma D$ بر خاک وارد می شود و خاک به طور خالص قادر به تحمل

نیروی معادل ($q_{ult} = \bar{q}$) به طور خالص از طریق پی (یا ستون) خواهد بود. بنابراین ظرفیت باربری

نهایی خاک به طور خالص برابر ($q_{ult} = \bar{q}$) است و این مقدار را باید بر ضریب اطمینان تقسیم نمود.



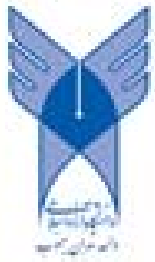
تذکر (۳): مهندسانی که رابطه (۱۰-۳-ج) را می پسندند عقیده ای شبیه گروه قبل دارند. با این تفاوت که معتقدند تنشی که بر تراز کف پی وارد می شود مشمول ضریب اطمینان نمی گردد و می بایست به طور کامل از ظرفیت باربری مجاز $(\frac{q_{ult}}{S.F.})$ کم شود. استدلالی که این گروه به کار می برند آن است که زمین قبل از خاکبرداری باری معادل با $\bar{q} = \gamma D$ را به خوبی تحمل می کرده است. پس اعمال ضریب اطمینان بر این بار معلوم منطقی نیست.

تذکر (۴): انتخاب ضریب اطمینان مناسب نیاز به تجربه و شناخت عمیق از خاک و سازه دارد. ضریب اطمینان کم سازه را با خطر مواجه می کند و ضریب اطمینان زیاد باعث غیراقتصادی شدن طرح می گردد. در هر حال مهندسان معمولاً ضریب اطمینان ظرفیت باربری نهایی (S.F.) را حداقل برابر 2.5-3.0 در نظر می گیرند.

$$S.F. \geq 2.5 - 3.0$$

تذکر (۵): روابط (۱۰-۳) به ترتیب از بالا به پائین محافظه کارانه تر می شوند. یعنی استفاده از رابطه (ج) محافظه کارانه تر (یا در جهت اطمینان تر) از رابطه (الف) است.

تذکر (۶): استفاده از هر یک از روابط (الف) تا (ج) در معادله (۱۰-۳) بنا به تشخیص مهندس و با



۲-۴-۱۰ اعمال ضریب اطمینان بر c و ϕ'

جمعی از محققان معتقدند که اعمال ضریب اطمینان بر پارامترهای مقاومتی خاک (یعنی C و Φ) اصولی تر و مناسب تر است. "مایرهوف" و "هنسن" جزو این دسته از محققان هستند. تحقیقات نشان می دهد که برای رسیدن به ضریب اطمینانی معادل ۳ تا ۴ بر روی q_{ult} ، ضریب اطمینان برشی معادل $S.F._{shear} = 1.4 - 1.6$ کافی و مناسب است. بنابراین برای بدست آوردن q_{all} از این روش به طریق زیر عمل می کنیم:

۱- ابتدا مقادیر چسبندگی و زاویه اصطکاک طراحی (ϕ'_d, c_d) را از روابط (۴-۱۰) محاسبه می

کنیم:

$$c_d = \frac{c}{S.F._{shear}} \quad (۴-۱۰-الف)$$

$$\phi'_d = \tan^{-1} \frac{\tan \phi'}{S.F._{shear}} \quad (۴-۱۰-ب)$$

که در آن

c_d و $\phi'_d =$ به ترتیب چسبندگی طراحی و زاویه اصطکاک طراحی

و $\phi' =$ به ترتیب چسبندگی و زاویه اصطکاک بدست آمده از آزمایش خاک

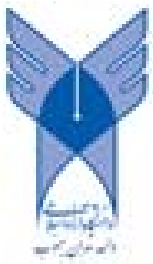
$S.F._{shear} =$ ضریب اطمینان برشی که بین ۱.۴ تا ۱.۶ انتخاب می شود.

۲- مقادیر c_d و ϕ'_d را در یکی از روابط تعیین ظرفیت باربری که در بخش بعد خواهد آمد قرار

داده و q_{all} را محاسبه می کنیم.

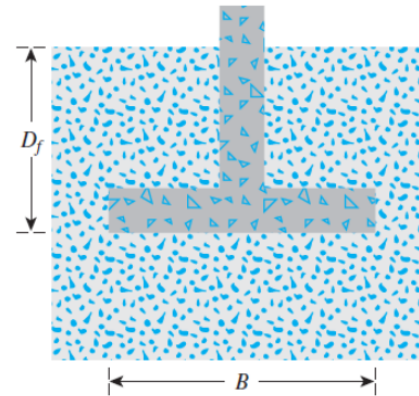
تذکره: در این روش برای محاسبه ظرفیت باربری خالص مجاز می توان از رابطه زیر استفاده کرد:

$$q_{all (net)} = q_{all} - \bar{q} \quad (۱۰-۵)$$



مثال

مثال: در یک شالوده مربعی مطابق، مطلوب است محاسبه ظرفیت باربری مجاز شالوده با استفاده از رابطه ترازقی در هر یک از دو حالت زیر»
 الف) فرض کنید ضریب اطمینان کلی بر روی q_{ult} برابر است با $S.F=3.0$ و ظرفیت باربری مجاز را از روابط (۳-۱۰) محاسبه کنید.
 ب) فرض کنید ضریب اطمینان برشی برابر است با $S.F_{shear} = 1.5$ و از روابط (۴-۱۰) استفاده کنید. کلیه اطلاعات لازم در شکل ارائه شده اند.



$$\begin{aligned}\phi' &= 30^\circ \\ c &= 10 \text{ kN/m}^2 \\ B = L &= 2.0 \text{ m} \\ \gamma &= 17 \text{ kN/m}^3 \\ D &= 1.0 \text{ m}\end{aligned}$$

پاسخ: الف) برای محاسبه ظرفیت باربری نهایی شالوده با استفاده از رابطه ی (۶-۱۰) ابتدا ضرایب ظرفیت باربری را با استفاده از جدول (۱-۱۰) محاسبه می کنیم:

$$\Phi = 30^\circ \xrightarrow{\text{جدول (1-10)}} N_c = 37.16; \quad N_q = 22.46; \quad N_\gamma = 19.13$$

با توجه به اینکه پی مربعی شکل است، داریم:

$$S_c = 1.3; \quad S_\gamma = 0.8$$

حال با توجه به رابطه (۶-۱۰) داریم:

$$\begin{aligned}q_u &= CN_c S_c + \bar{q} N_q + 0.5 \gamma B N_\gamma S_\gamma \\ q_u &= (10)(37.16)(1.3) + (17 \times 1)(22.46) \\ &\quad + 0.5(17)(2)(19.13)(0.8) \\ q_u &= 1127 \text{ KN/m}^2\end{aligned}$$

با استفاده از روابط (۳-۱۰) داریم:
 رابطه (۳-۱۰) الف):



مثال

$$q_{all} = \frac{q_{ult}}{S.F.} = \frac{1127}{3} = 376 \text{ kN/m}^2$$

رابطه (۱۰-۳-ب):

$$q_{all} = \frac{q_{ult} - \bar{q}}{S.F.} = \frac{1127 - (17 \times 1)}{3} = 370 \text{ kN/m}^2$$

رابطه (۱۰-۳-ج):

$$q_{all} = \frac{q_{ult}}{S.F.} - \bar{q} = \frac{1127}{3} - (17 \times 1) = 359 \text{ kN/m}^2$$

ب) با استفاده از روابط (۱۰-۴) داریم:

$$c_d = \frac{c}{S.F_{shear}} = \frac{10}{1.5} = 6.67 \text{ kN/m}^2$$

$$\varphi'_d = \tan^{-1} \left(\frac{\tan \varphi'}{S.F_{shear}} \right) = \tan^{-1} \left(\frac{\tan 30^\circ}{1.5} \right) = 21^\circ$$

حال با استفاده از مقدار φ_d و با توجه به جدول (۱۰-۱) داریم:

$$\varphi'_d = 21^\circ \xrightarrow{\text{جدول (1-10)}} N_c = 18.92; \quad N_q = 8.26; \quad N_\gamma = 4.31$$

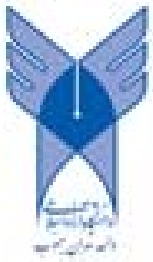
سپس مقدار ظرفیت باربری مجاز را با استفاده از رابطه ترزاقی محاسبه می کنیم:

$$\begin{aligned} q_{all} &= cN_cS_c + \bar{q}N_q + 0.5\gamma BN_\gamma S_\gamma \\ q_{all} &= (6.67)(18.92)(1.3) + (17 \times 1)(8.26) \\ &\quad + 0.5(17)(2)(4.31)(0.8) \end{aligned}$$

$$q_{all} = 363 \text{ kN/m}^2$$

لازم به ذکر است که با قراردادن مقادیر c_d و φ_d در رابطه ترزاقی به جای q_{ult} مقدار q_{all} محاسبه

می شود.



مثال

مثال: برای یک پی با باری معادل $P=170 \text{ ton}$ که بر یک لایه خاک با مشخصات زیر اجرا می شود شالوده ای دایره ای طرح نمائید. فرض کنید که پی در عمق $D=1.0 \text{ m}$ از سطح زمین اجرا خواهد شد.

مشخصات خاک که از آزمایش سه محوری u بدست آمده عبارتند از:

$$c = 5 \text{ ton/m}^2; \quad \varphi' = 22^\circ; \quad \gamma = 106 \text{ ton/m}^3$$

ضریب اطمینان را برابر $S.F=3.5$ فرض کنید.

پاسخ: با استفاده از جدول (۱۰-۱) داریم:

$$\varphi' = 22^\circ \xrightarrow{\text{جدول (1-10)}} N_c = 20.27; \quad N_q = 9.19; \quad N_\gamma = 5.09$$

در یک پی دایره ای $S_c = 1.3$ و $S_\gamma = 0.6$ است. بنابراین با نوشتن رابطه ظرفیت باربری ترازقی با فرض آنکه نظر پی برابر B باشد، داریم:

$$q_{ult} = cN_cS_c + \bar{q}N_q + 0.5\gamma BN_\gamma S_\gamma$$

$$q_{ult} = (5)(20.27)(1.3) + (1.6 \times 1)(9.19) + 0.5(1.6)(B)(5.09)(0.6)$$

$$q_{ult} = 146.5 + 2.44B$$

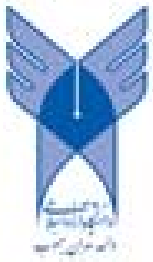
و ظرفیت باربری مجاز برابر است با:

$$q_{all} = \frac{q_{ult}}{S.F.} = \frac{146.5 + 2.44B}{3.5} = 41.86 + 0.698B \quad (\text{ton/m}^2)$$

حال با آزمودن مقادیر مختلف برای قط پی (B)، بهترین مقدار را برای آن انتخاب می کنیم. این محاسبه در جدول زیر خلاصه شده است.

نتیجه	تنش موجود در زیر پی $q = \frac{P}{A}$	مساحت پی $A = \frac{\pi B^2}{4}$ (m^2)	$=41.86+0.698Bq_{all}$ (ton/m^2)	قطر پی (B) (m)
غیرقابل قبول	۹۶.۲	۱.۷۷	۴۲.۹	۱.۵
غیرقابل قبول	۵۴.۱	۳.۱۴	۴۳.۳	۲.۰
*قابل قبول و اقتصادی	۴۲.۷	۳.۹۸	۴۳.۴	۲.۲۵
غیراقتصادی	۳۴.۶	۴.۱۹	۴۳.۶	۲.۵

بنابراین قطر پی برابر با $B=2.25 \text{ m}$ انتخاب می شود. دقت کنید که ملاک انتخاب پی مناسب در مرحله اول آن است که تنش موجود در زیر پی کمتر از تنش مجاز باشد یعنی ($q \leq q_{all}$) و در مرحله دوم باید فاصله این دو مقدار، مینیموم باشد تا طرح از نظر اقتصادی مقرون به صرفه باشد. مثلاً در این مثال اگرچه می توان نظر پی را برابر 2.5 متر در نظر گرفت، اما این ابعاد برای پی بیش از حد مورد نیاز و در نتیجه غیراقتصادی است.



مکانیسم مایر هوف برای گسیختگی

۱- گوه زیر پی

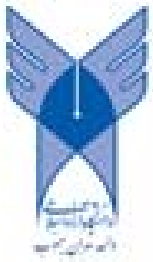
• زاویه گوه $(45^\circ + \phi'/2)$

۲- ناحیه پلاستیک (برش شعاعی)

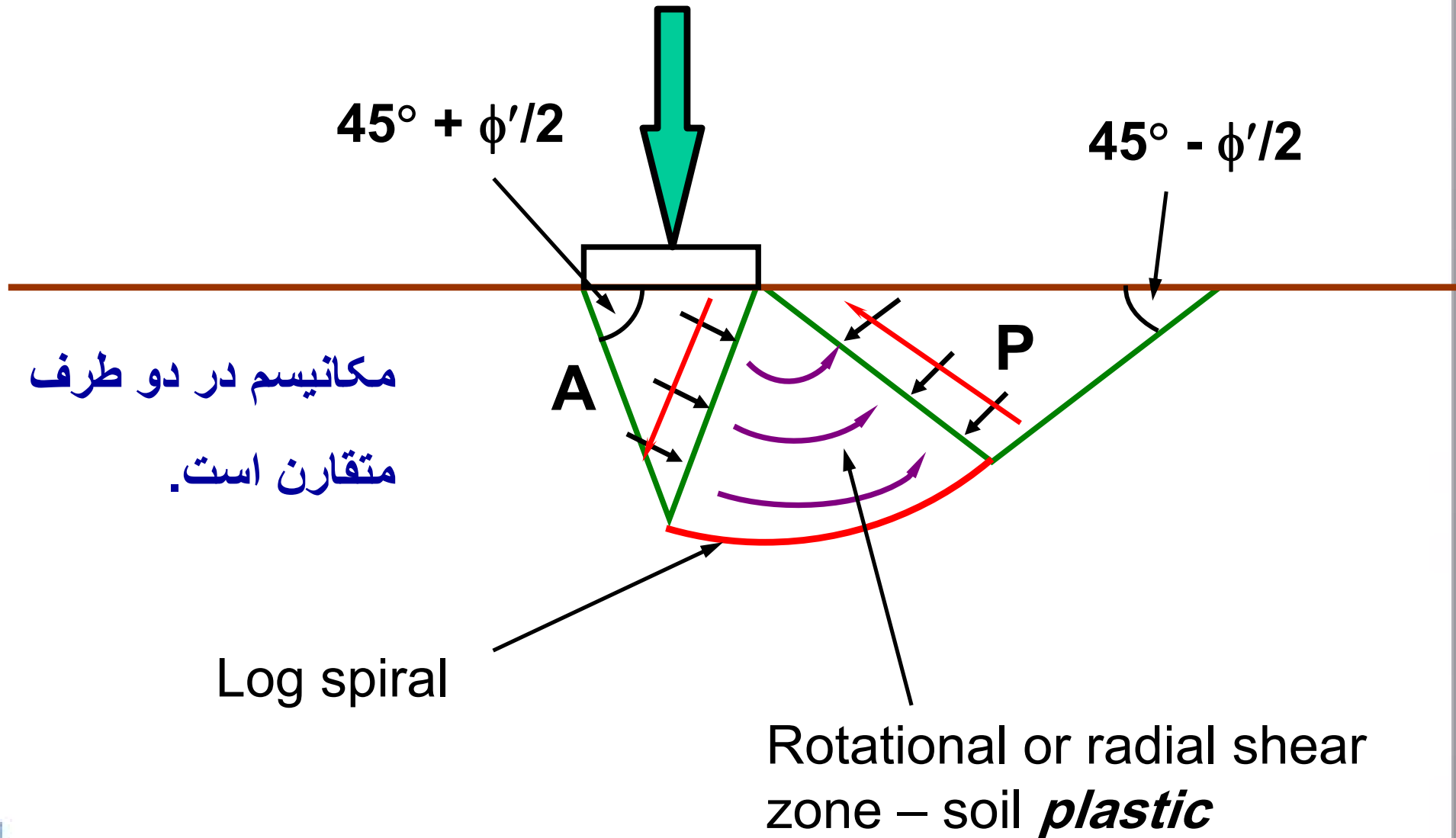
• اسپیرال لگاریتم

۳- ناحیه پاسیو

• زاویه گوه $(45^\circ - \phi'/2)$



مکانیسم مایر هوف برای گسیختگی



مکانیسم مایر هوف برای گسیختگی

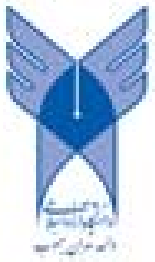
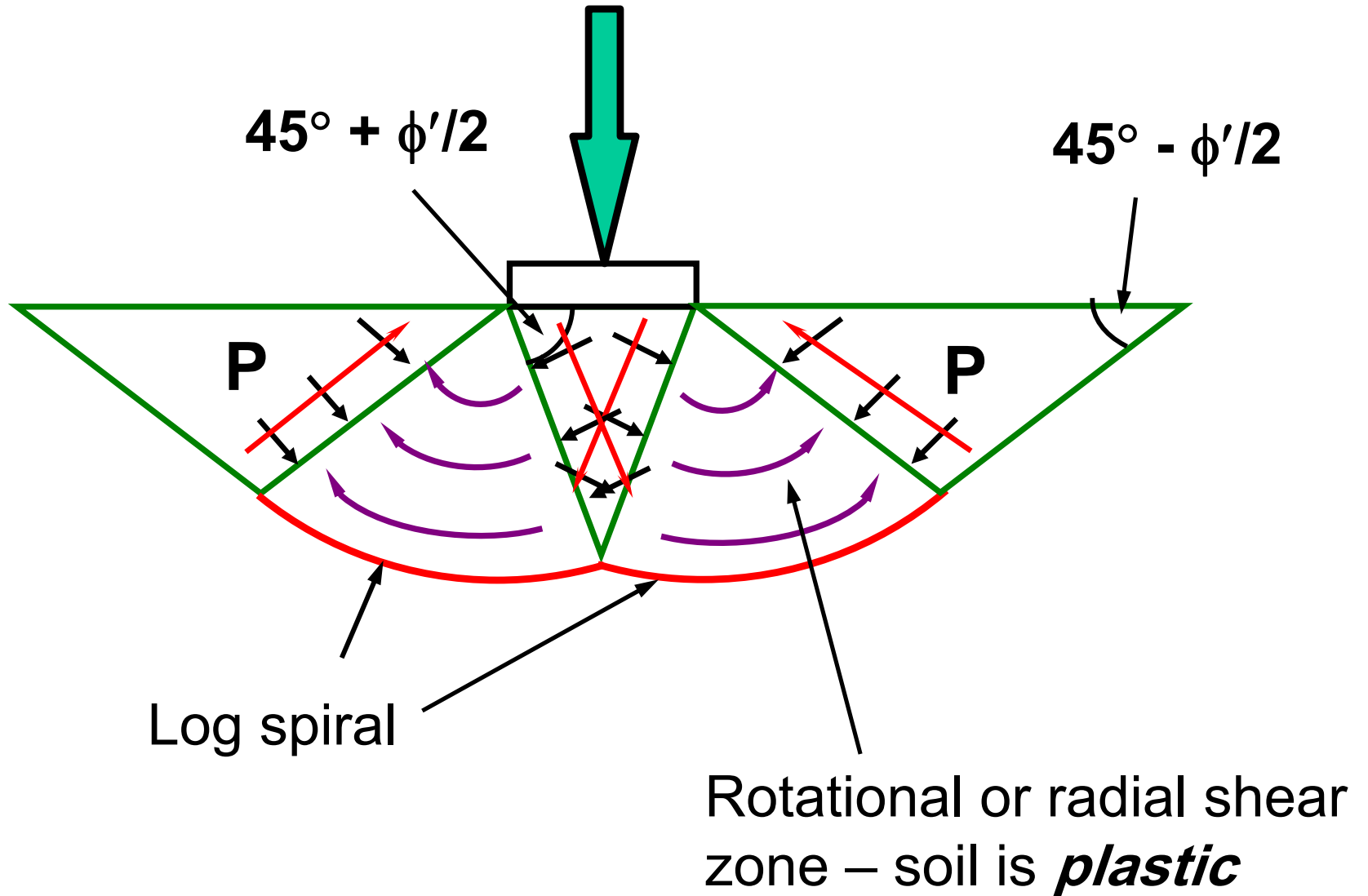


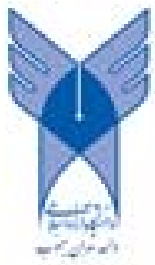
Table 12.3 Shape, depth and load inclination factors of Meyerhof, Hansen and Vesic

Factors	Meyerhof	Hansen	Vesic
s_c	$1 + 0.2N_\phi \frac{B}{L}$	$1 + \frac{N_q}{N_c} \frac{B}{L}$	The shape and depth factors of Vesic are the same as those of Hansen.
s_q	$1 + 0.1N_\phi \frac{B}{L}$ for $\phi > 10^\circ$	$1 + \frac{B}{L} \tan \phi$	
s_γ	$s_\gamma = s_q$ for $\phi > 10^\circ$ $s_\gamma = s_q = 1$ for $\phi = 0$	$1 - 0.4 \frac{B}{L}$	
d_c	$1 + 0.2\sqrt{N_\phi} \frac{D_f}{B}$	$1 + 0.4 \frac{D_f}{B}$	
d_q	$1 + 0.1\sqrt{N_\phi} \frac{D_f}{B}$ for $\phi > 10^\circ$	$1 + 2 \tan \phi (1 - \sin \phi)^2 \frac{D_f}{B}$	
d_γ	$d_\gamma = d_q$ for $\phi > 10^\circ$ $d_\gamma = d_q = 1$ for $\phi = 0$	1 for all ϕ	
		Note; Vesic's s and d factors = Hansen's s and d factors	
i_c	$1 - \frac{\alpha^\circ}{90}^2$ for any ϕ	$i_q - \frac{1 - i_q}{N_q - 1}$ for $\phi > 0$	Same as Hansen for $\phi > 0$
		$0.5 \left(1 - \frac{Q_h}{A_f c_a}\right)^{\frac{1}{2}}$ for $\phi = 0$	$1 - \frac{mQ_h}{A_f c_a N_c}$
i_q	$i_q = i_c$ for any ϕ	$1 - \frac{0.5Q_h}{Q_u + A_f c_a \cot \phi}^5$	$1 - \frac{Q_h}{Q_u + A_f c_a \cot \phi}^m$
i_γ	$1 - \frac{\alpha^\circ}{\phi}^2$ for $\phi > 0$ $i_\gamma = 0$ for $\phi = 0$	$1 - \frac{0.7Q_h}{Q_u + A_f c_a \cot \phi}^5$	$1 - \frac{Q_h}{Q_u + A_f c_a \cot \phi}^{m+1}$

$$N_\gamma = (N_q - 1) \tan(1.4\phi) \quad (\text{Meyerhof})$$

$$N_\gamma = 1.5(N_q - 1) \tan \phi \quad (\text{Hansen})$$

$$N_\gamma = 2(N_q + 1) \tan \phi \quad (\text{Vesic})$$

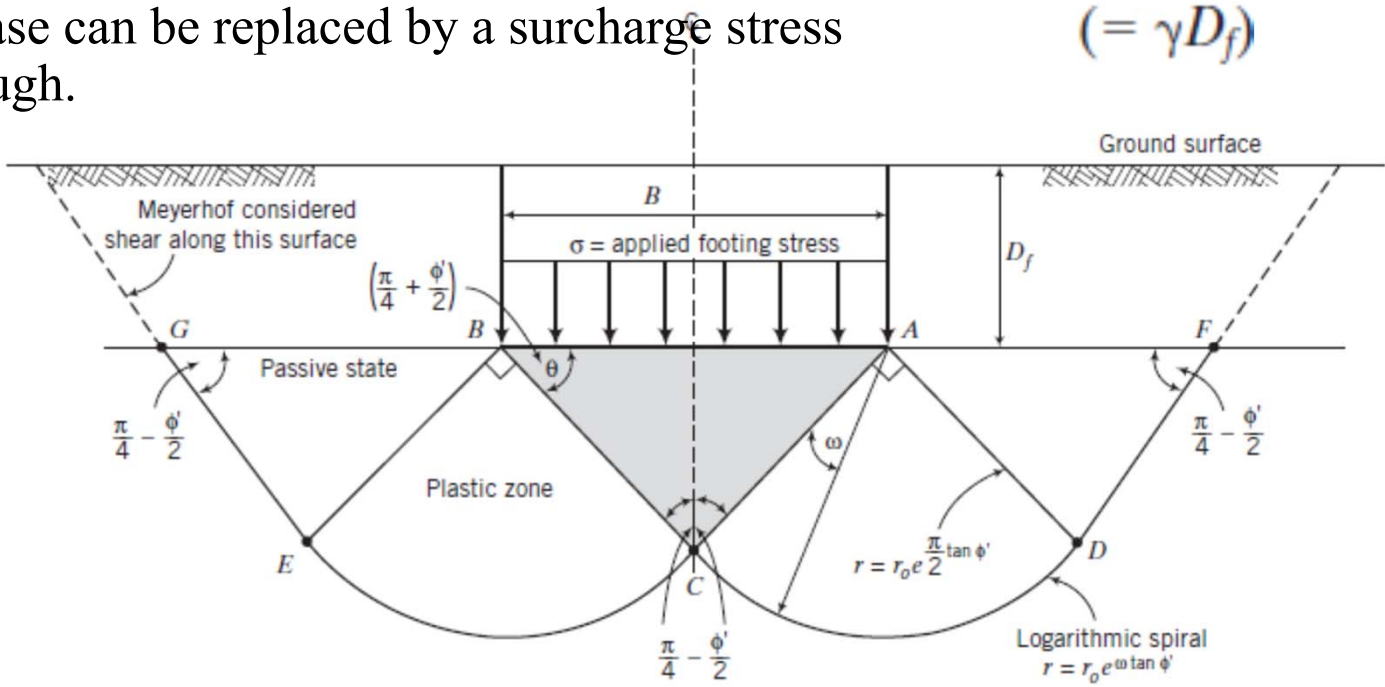


BEARING CAPACITY EQUATIONS

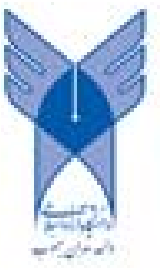
- 1. The soil is a semi-infinite, homogeneous, isotropic, weightless, rigid-plastic material.
- 2. The embedment depth is not greater than the width of the footing $(D_f < B)$
- 3. General shear failure occurs.
- 4. The angle θ in the wedge is $\theta = 45^\circ + \phi'/2$. After, it was found (Vesic, 1973) that

$$\theta = 45^\circ + \phi'/2$$

- 5. The shear strength of the soil above the footing base is negligible. Later, Meyerhof (1951) considered the shear resistance above the footing base.
- 6. The soil above the footing base can be replaced by a surcharge stress $(= \gamma D_f)$
- 7. The base of the footing is rough.



Conventional failure surface.



Geometric Factors for Use in Theoretical Bearing Capacity Equations

Geometric parameters for TSA

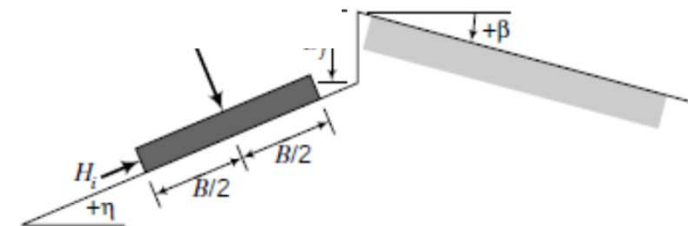
s_c	d_c	i_c	b_c	g_c
$1 + 0.2 \frac{B'}{L'}$	$1 + 0.33 \tan^{-1} \frac{D_f}{B'}$ See note 1	$1 - \frac{nH}{5.14 s_u B' L'}$ See note 2	$1 - \frac{\eta^\circ}{147}$ $\beta < \phi'_p; \eta^\circ + \beta^\circ < 90^\circ$ See Figure 12.9	$1 - \frac{\beta^\circ}{147}$ $\beta < \phi'_p; \eta^\circ + \beta^\circ < 90^\circ$ See Figure 12.9

Geometric parameters for ESA

s_q	d_q	i_q	b_q	g_q
$1 + \frac{B'}{L'} \tan \phi'_p$	$1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \left(\frac{D_f}{B'} \right)$	$\left(1 - \frac{H}{V_n} \right)^n$ See note 2	$(1 - \eta \tan \phi'_p)^2$ η is in radians	$(1 - \tan \beta)^2$
s_γ	d_γ	i_γ	b_γ	g_γ
$1 - 0.4 \frac{B'}{L'}$	1	$\left(1 - \frac{H}{V_n} \right)^{n+1}$ See note 2	$b_\gamma = b_q$	$g_\gamma = g_q$

Note 1: If the shear strength of the soil above the footing is low compared with that of the soil below the footing, you should set all depth factors to 1. The term $\tan^{-1} \left(\frac{D_f}{B'} \right)$ is in radians.

Note 2: The depth and shape factors for inclined loads should be set to 1. For loading inclined in the direction of the width, B , $\theta = 90^\circ$ in Figure 12.10d, $n = n_B = \left(2 + \frac{B'}{L'} \right) / \left(1 + \frac{B'}{L'} \right)$. For loading inclined in the direction of the length, L , $\theta = 0^\circ$ in Figure 12.10d, $n = n_L = \left(2 + \frac{L'}{B'} \right) / \left(1 + \frac{L'}{B'} \right)$. For other loading conditions, $n = n_L \cos^2 \theta + n_B \sin^2 \theta$.



اثر سفره آب زیر زمینی

$$\gamma_e = \gamma' + \frac{d}{H} (\gamma - \gamma') \quad (۱۴-۱۰-الف)$$

$$\gamma_e = (2H - d) \frac{d}{H^2} \gamma + \frac{\gamma'}{H^2} (H - d)^2 \quad (۱۴-۱۰-ب)$$

که در آن

$\gamma_e =$ وزن مخصوص معادل

$\gamma' = \gamma_{sat} - \gamma_w$ وزن مخصوص غوطه ور (یا موثر)

$d =$ عمق تراز آب زیرزمینی نسبت به کف پی

$H =$ عمق گوه مثلثی مطابق رابطه (۱۰-۱۳)

تذکر (۱): استفاده از هر یک از روابط (۱۰-۱۴) بسته به میل طراح مجاز است.

تذکر (۲): هر دو رابطه (۱۰-۱۴) نشان می دهند که اگر $d=0$ باشد یعنی تمام گوه اشباع شده است

و در این حالت $\gamma_e = \gamma'$ و در حالتی که $d=H$ باشد یعنی تمام گوه غیراشباع است و در این حالت

$$\gamma_e = \gamma$$

تذکر (۳): برخی از مولفان معتقدند که به جای عمق گوه (H) باید عرض پی (B) در رابطه (۱۰-۱۴)

(۱۴) مورد استفاده قرار گیرد.

حالت سوم: تراز آب زیرزمینی بالای کف پی است.

در این حالت گوه تماماً اشباع است و می بایست برای وزن مخصوص خاک در جمله سوم ظرفیت باربری

از γ' استفاده شود. همچنین در جمله سربار (جمله دوم) به جای \bar{q} باید تنش موثر وارد بر تراز کف پی

استفاده شود. بنابراین در این حالت اعمال اصلاحات زیر ضروری است.

در جمله دوم: $\bar{q} = \gamma(H - d) + \gamma'd$ تنش موثر وارد بر تراز کف پی

در جمله سوم: $\gamma = \gamma'$

تذکر: همانطور که دیدیم، عمق H تعیین کننده سه حالت مختلف فوق بود. برخی از مولفان معتقدند

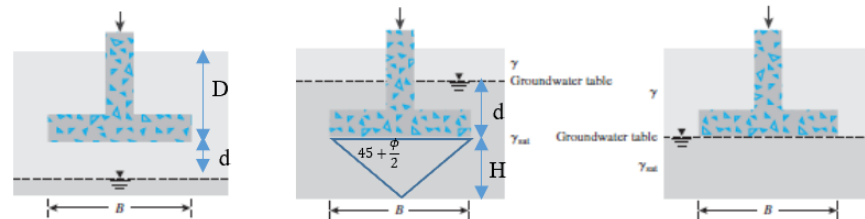
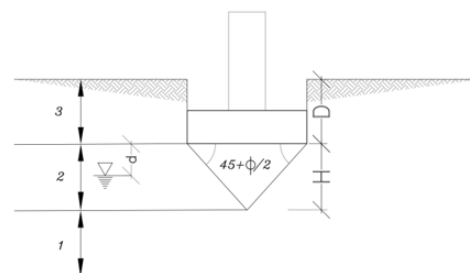
به جای عمق H می توان عرض پی B را ملاک تفکیک حالات مختلف تراز آب تعیین کرد.

$$q_{ult} = cN_c S_c \dots + \bar{q}N_q S_q \dots + 0.5\gamma B N_\gamma S_\gamma \dots$$

تراز آب زیرزمینی همکف است در یکی از حالات زیر قرار بگیرد:

حالت اول: تراز آب در زیر گوه مثلثی قرار دارد ($d > H$)

در این حالت با توجه به اینکه تراز آب وارد منطقه گسیختگی در خاک نشده است، توصیه می شود که هیچ اصلاحی در رابطه ظرفیت باربری در نظر گرفته نشود. به عبارت دیگر اگر آب زیرزمینی در عمق زیر گوه قرار داشته باشد، تاثیری در ظرفیت باربری ندارد.



شکل ۲۰-۱۰ حالات مختلف قرارگیری تراز آب در زیر پی و معرفی پارامترهای روابط اصلاح ظرفیت باربری خاک تحت تاثیر تغییر تراز آب زیرزمینی

حالت دوم: تراز آب زیر زمینی در ناحیه گوه مثلثی قرار دارد ($0 < d < H$)

فرض می کنیم تراز آب نسبت به تراز کف پی مطابق شکل (۱۰-۲۰) برابر با d باشد در این حالت پیشنهاد

می شود که مقدار γ در جمله سوم معادله ظرفیت باربری (جمله هندسی) با مقدار وزن مخصوص معادل

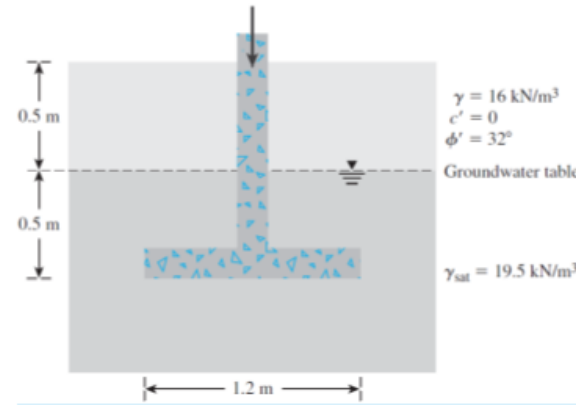
(γ_e) جایگزین شود. وزن مخصوص معادل خاک در واقع نوعی انترپولاسیون بین حالت مرطوب و اشباع

خاک است که با توجه به مقدار d تعیین می شود. برای محاسبه وزن مخصوص معادل، روابط زیر پیشنهاد

شده است:



مثال: یک شالوده مربعی مطابق شکل مفروض است. اگر ضریب اطمینان برابر ۳ فرض شود، مقدار بار ناخالص که این شالوده تحمل می کند را با استفاده از روش مایرهورف تعیین کنید.



پاسخ: با توجه به $\phi = 32^\circ$ از روابط مایرهورف داریم:

$$N_q = 23.18; \quad N_\gamma = 22.02$$

$$k_p = \tan^2 \left(45 + \frac{\phi}{2} \right) = \tan^2 \left(45 + \frac{32}{2} \right) = 3.25$$

$$S_q = S_\gamma = 1 + 0.1 k_p \frac{B}{L} = 1 + 0.1(3.25) \frac{1.2}{1.2} = 1.325$$

$$d_q = d_\gamma = 1 + 0.1 \sqrt{k_p} \frac{B}{L} = 1 + 0.1 \sqrt{3.25} \frac{1.2}{1.2} = 1.15$$

$$\bar{q} = (16 \times 0.5) + (19.5 - 9.81)(0.5) = 12.845 \text{ kN/m}^2$$

چون سطح آب زیرزمینی در بالای زمین قرار دارد، پس در جمله سوم به جای γ ، از $\gamma' = \gamma_{sat} - \gamma_w$ استفاده می کنیم. با توجه به اینکه $c=0$ است، رابطه مایرهورف به صورت زیر در می آید:

$$q_{ult} = \bar{q} N_q S_q d_q + 0.5 \gamma' B N_\gamma S_\gamma d_\gamma$$

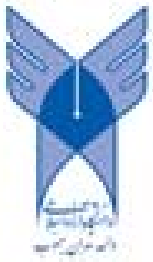
$$q_{ult} = (12.845)(23.18)(1.325)(1.15) + 0.5(19.5 - 9.81)(1.2)(22.02)(1.325)(1.15)$$

$$q_{ult} = 648.8 \text{ kN/m}^2$$

$$q_{all} = \frac{q_{ult}}{SF} = \frac{648.8}{3} = 216.3 \text{ kN/m}^2$$

و بنابراین مقدار بار ناخالص مجاز برابر است با:

$$Q_{all} = q_{all} B^2 = 216.3 (1.2)^2 = 311.5 \text{ kN}$$



مثال: یک پی مربعی با بار قائم مطابق شکل مفروض است. تراز آب در عمق ۰.۵ متر قرار گرفته است مطلوب است محاسبه ظرفیت باربری پی با فرض $SF=3$ ، با استفاده از روش هنسن

$$B = L = 2.5m$$

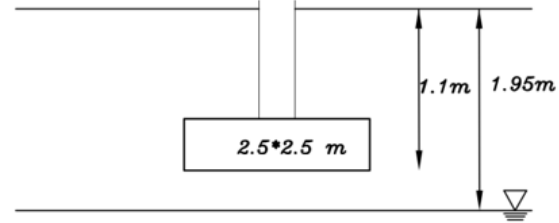
مشخصات خاک عبارتند از

$$\phi' = 32^\circ \quad c = 0$$

$$\gamma = 16kN/m^3$$

$$\gamma_{sat} = 19.5kN/m^3$$

$$G_s = 2.66$$



$$S_\gamma = 1 - 0.4 \frac{B'}{L'} = 0.6$$

$$d_q = 1 + 2 \tan \phi (1 - \sin \phi)^2 \frac{D}{B} = 1.11$$

$$d_\gamma = 1.1$$

با استفاده از فرمول هنسن و با توجه به اینکه $c=0$ است، داریم:

$$q_{ult} = \bar{q} N_q S_q d_\gamma + 0.5 \gamma' B N_\gamma S_\gamma d_\gamma$$

$$q_{ult} = (18.1 \times 1.1)(33)(1.57)(1.11) + 0.5(14.86)(2.5)(34)(0.6)(1.1)$$

$$q_{ult} = 1524 \text{ kPa}$$

$$q_{all} = \frac{q_{ult}}{SF} = \frac{1524}{3} = 508 \text{ kPa}$$

این مقدار برای ظرفیت باربری به نظر زیاد و کمی غیرطبیعی است. به همین دلیل باید با احتیاط بسیار زیاد از آن استفاده شود.

پاسخ: ابتدا وزن مخصوص موثر خاک را با استفاده از رابطه (۱۰-۱۴-الف) محاسبه می کنیم:

$$\gamma_d = \frac{\gamma}{1 + \omega} = \frac{18.1}{1 + 0.1} = 16.45 \text{ KN/m}^2$$

$$\gamma_d = \frac{\gamma_w G_s}{1 + e} \Rightarrow 16.45 = \frac{(9.81)(2.68)}{1 + e} \Rightarrow e = 0.6$$

$$\gamma_{sat} = \frac{\gamma_w (G_s + e)}{1 + e} = \frac{9.81(2.68 + 0.6)}{1 + 0.6} = 20.1 \text{ KN/m}^3$$

$$H = 0.5 B \tan \left(45 + \frac{\phi}{2} \right) = 0.5(2.5) \tan \left(45 + \frac{35}{2} \right) = 2.4 \text{ m}$$

$$d = 1.95 - 1.1 = 0.85 \text{ m}$$

$$\gamma_e = (2H - d) \frac{d}{H^2} \gamma + \frac{\gamma'}{H^2} (H - d)^2$$

$$\gamma_e = (2 \times 2.4 - 0.85) \frac{0.85}{2.4^2} 18.1 + \frac{20.1 - 9.81}{2.4^2} (2.4 - 0.85)^2$$

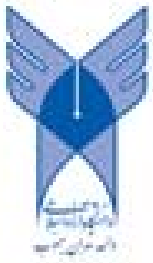
$$\gamma_e = 14.85 \text{ KN/m}^2$$

ون پی مربع است، مقدار زاویه اصطکاک خاک را بر اساس توصیه باواز و مایرهوف (روابط ۱۰-۹-الف) و (۱۰-۱۱) اصلاح نمی کنیم. با داشتن $\phi = 35^\circ$ داریم:

$$N_q = 33; \quad N_\gamma = 34$$

$$2 \tan \phi (1 - \sin \phi)^2 = 2 \tan 35 (1 - \sin 35)^2 = 0.255$$

$$S_q = 1 + \frac{B'}{L'} \sin \phi = 1.57$$



EXAMPLE 12.1 Allowable Bearing Capacity of a Sand

A footing 2 m square, subjected to a centric vertical load, is located at a depth of 1.0 m below the ground surface in a deep deposit of compacted sand ($\phi'_p = 35^\circ$, $\phi'_{cs} = 30^\circ$, and $\gamma_{sat} = 18 \text{ kN/m}^3$). The groundwater level is 5 m below the ground surface, but you should assume that the soil above the groundwater is saturated. The friction angles were obtained from plain strain tests. Determine the allowable bearing capacity for a factor of safety of 3.

Strategy It is a good policy to sketch a diagram illustrating the conditions given (see Figure E12.1). The groundwater level is located at $(5 \text{ m} - 1 \text{ m}) = 4 \text{ m}$ from the footing base. That is, the groundwater level is more than $B = 2 \text{ m}$ below the base. We can neglect the effects of groundwater.

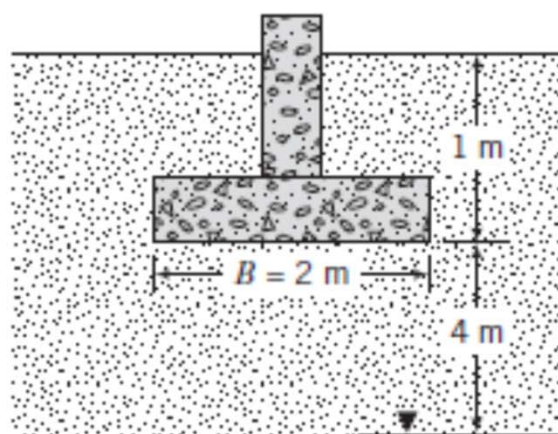


FIGURE E12.1

Solution 12.1

Step 1: Calculate the bearing capacity factors and geometric factors. Assume rough footing. Use $\phi' = \phi'_p = 35^\circ$.

The eccentricity is zero, so $B' = B$ and $L' = L$.

$$N_q = e^{\pi \tan \phi'_p} \tan^2 \left(45^\circ + \frac{\phi'_p}{2} \right) = e^{\pi \tan 35^\circ} \tan^2 \left(45^\circ + \frac{35^\circ}{2} \right) = 33.3$$

$$N_q - 1 = 32.3$$

$$N_\gamma = 0.1054 \exp(9.6 \phi'_p) = 0.1054 \exp \left(9.6 \times \frac{35 \times \pi}{180} \right) = 37.1$$

$$s_q = 1 + \frac{B'}{L'} \tan \phi'_p = 1 + \frac{2}{2} \tan 35^\circ = 1.70$$

$$s_\gamma = 1 - 0.4 \frac{B'}{L'} = 1 - 0.4 \frac{2}{2} = 0.6$$

$$d_\gamma = 1.0$$

$$d_q = 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \left(\frac{D_f}{B'} \right)$$

$$= 1 + (2 \tan 35^\circ) (1 - \sin 35^\circ)^2 \left[\tan^{-1} \left(\frac{1}{2} \right) \times \frac{\pi}{180} \right] = 1.13$$

Step 2: Calculate the ultimate net bearing capacity.

$$q_u = \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

$$q_u = (18 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 18 \times 2 \times 37.1 \times 0.6 \times 1.0)$$

$$= 1515 \text{ kPa}$$

$$q_a = \frac{q_u}{\text{FS}} + \gamma D_f$$

$$= \frac{1515}{3} + 18 \times 1 = 523 \text{ kPa}$$



EXAMPLE 12.2 *The Effects of Groundwater on Bearing Capacity*

Compare the ultimate net bearing capacity for Example 12.1 using $\phi'_p = 35^\circ$ when the groundwater is located (a) at 5 m below the ground surface, (b) at the ground surface, (c) at the bottom of the base of the footing, and (d) at 1 m below the base.

Strategy The trick here is to use the appropriate value of the unit weight in the bearing capacity equation.

Solution 12.2

Step 1: Calculate bearing capacity numbers and shape and depth factors. These values are the same as in Example 12.1.

Step 2: Substitute values from Step 1 into Equation (12.14).

(a) *Groundwater level at 5 m below the surface.* The groundwater level is 4 m below the base, which is greater than the width of the footing. Therefore, groundwater has no effect.

$$\text{From Example 12.1: } q_u = 1515 \text{ kPa}$$

(b) *Groundwater level at the ground surface.* In this case, the groundwater level will affect the bearing capacity. You should use

$$\gamma' = \gamma_{sat} - \gamma_w = 18 - 9.8 = 8.2 \text{ kN/m}^3$$

$$\begin{aligned} q_u &= \gamma' D_f (N_q - 1) s_q d_q + 0.5 \gamma' B' N_\gamma s_\gamma d_\gamma \\ &= (8.2 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 8.2 \times 2 \times 37.1 \times 0.6 \times 1.0) \\ &= 691 \text{ kPa} \end{aligned}$$

Alternatively, since the change in the unit weight is the same for both terms of the bearing capacity equation, we can simply find q_u by taking the ratio γ'/γ , that is,

$$q_u = 1515 \times \frac{8.2}{18} = 690 \text{ kPa}$$

- (c) *Groundwater level at the bottom of the base.* In this case, the groundwater level will affect the last term in the bearing capacity.

Thus,

$$\begin{aligned} q_u &= \gamma D_f(N_q - 1)s_q d_q + 0.5\gamma' B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 8.2 \times 2 \times 37.1 \times 0.6 \times 1.0) \\ &= 1299 \text{ kPa} \end{aligned}$$

- (d) *Groundwater level at 1 m below the bottom of the base.* In this case, the groundwater level is within a depth B below the base and will affect the last term in the bearing capacity, where you should use

$$\gamma' B' = \gamma_{sat} z + \gamma'(B' - z) = 18 \times 1 + 8.2 \times (2 - 1) = 26.2 \text{ kN/m}^2$$

Thus,

$$\begin{aligned} q_u &= \gamma D_f(N_q - 1)s_q d_q + 0.5(\gamma' B') N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 26.2 \times 37.1 \times 0.6 \times 1.0) \\ &= 1408 \text{ kPa} \end{aligned}$$

Step 3: Compare results.

We will compare the results by dividing (normalizing) each ultimate net bearing capacity by the ultimate net bearing capacity of case (a).

Groundwater level at	$\frac{q_u}{(q_u)_{(a)}} \times 100$
(b) Ground surface	$\frac{691}{1515} \times 100 \approx 46\%$
(c) Base	$\frac{1299}{1515} \times 100 = 86\%$
(d) 1 m below base	$\frac{1408}{1515} \times 100 = 93\%$

Note: $(q_u)_{(a)}$ is the net ultimate bearing capacity for case (a).

The groundwater level rising to the surface will reduce the bearing capacity by more than one-half.



EXAMPLE 12.3 Allowable Short-term Bearing Capacity of a Clay Soil

A footing $1.8 \text{ m} \times 2.5 \text{ m}$ is located at a depth of 1.5 m below the ground surface in a deep deposit of a saturated overconsolidated clay. The groundwater level is 2 m below the ground surface. The undrained shear strength from a direct simple shear test is 120 kPa and $\gamma_{\text{sat}} = 20 \text{ kN/m}^3$. Determine the allowable bearing capacity, assuming a factor of safety of 3, for short-term condition. Neglect the effects of embedment.

Strategy Use the equation for the short-term bearing capacity. You do not need to consider the effect of groundwater when you are evaluating short-term condition.

Solution 12.3

Step 1: Calculate geometric factors.

$$\text{No eccentricity: } B' = B, \quad L' = L$$

$$s_c = 1 + 0.2 \frac{B'}{L'} = 1 + 0.2 \frac{1.8}{2.5} = 1.14, \quad d_c = 1$$

Step 2: Calculate q_u .

$$q_u = 5.14 s_u s_c d_c = 5.14 \times 120 \times 1.14 \times 1 = 703 \text{ kPa}$$

Step 3: Calculate q_a .

$$q_a = \frac{q_u}{\text{FS}} + \gamma D_f = \frac{703}{3} + 1.5 \times 20 = 264 \text{ kPa}$$



Solution 12.4

Step 1: Calculate bearing capacity numbers, shape, and depth factors.

Assume $B' = B = 1.5$ m and $\frac{L'}{B'} = 1.5$; that is, $L' = 1.5 \times 1.5 = 2.25$ m and $\frac{B'}{L'} = \frac{1.5}{2.25} = 0.67$ m. Footing area $A = B' L' = 1.5 \times 2.25 = 3.375$ m².

$$(\phi'_p)_{ps} = \frac{9}{8}(\phi'_p)_{tr} = \frac{9}{8} \times 28.4^\circ = 32^\circ$$

$$N_q = e^{\pi \tan 32^\circ} \tan^2(45^\circ + 32^\circ/2) = 23.2$$

$$N_q - 1 = 23.2 - 1 = 22.2$$

Assume rough footing.

$$N_\gamma = 0.1054 \exp(9.6\phi'_p) = 0.1054 \exp\left(9.6 \times \frac{32 \times \pi}{180}\right)$$

$$= 22.5$$

$$s_q = 1 + \frac{B'}{L'} \tan \phi'_p = 1 + 0.67 \tan 32^\circ = 1.42$$

$$s_\gamma = 1 - 0.4 \frac{B'}{L'} = 0.73$$

$$d_q = 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \frac{D_f}{B'}$$

$$= 1 + (2 \tan 32^\circ)(1 - \sin 32^\circ)^2 \left[\tan^{-1} \left(\frac{1}{1.5} \right) \times \frac{\pi}{180} \right]$$

$$= 1.18$$

$$d_\gamma = 1.0$$

Step 2: Calculate the ultimate and allowable bearing capacity.

Substitute the values in Step 1 into the bearing capacity equation, Equation (12.16). The groundwater level is located more than B below the base. Therefore, groundwater will not affect the bearing capacity.

$$\begin{aligned}q_u &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 22.2 \times 1.42 \times 1.18) + (0.5 \times 18 \times 1.5 \times 22.5 \times 0.73 \times 1.0) = 891 \text{ kPa}\end{aligned}$$

$$q_{ult} = q_u + \gamma D_f = 891 + 18 \times 1 = 909 \text{ kPa}$$

$$q_a = \frac{q_u}{FS} + \gamma D_f = \frac{891}{3} + 18 \times 1 = 315 \text{ kPa}$$

$$R = q_{ult} \times A = 909 \times 3.375 = 3068 \text{ kN}$$

$$P_a = q_a \times A = 315 \times 3.375 = 1063 \text{ kN}$$

Step 3: Calculate the imposed stress based on ASD and LRFD.

$$\text{ASD: } P = \text{DL} + \text{LL} = 800 + 1000 = 1800 \text{ kN}$$

$$\text{LRFD: } P_{uf} = 1.25 \text{ DL} + 1.75 \text{ LL} = 1.25 \times 800 + 1.75 \times 1000 = 2750 \text{ kN}$$

The term P_{uf} is the factored load.

Step 4: Check suitability of assumed foundation size.

$$\text{LRFD: } \phi R = 0.8 \times 3068 = 2454 \text{ kN} < P_{uf} (= 2750 \text{ kN})$$

Unacceptable; try another footing size.

$$\text{ASD: } P_a (= 1063 \text{ kN}) < P (= 1800 \text{ kN})$$

Unacceptable; try another footing size.

Step 5: Try another width and recalculate.

We need to try a larger B , keeping $\frac{L'}{B'} = 1.5$. Try $B = B' = 2$ m. The depth factor for this case changes to 1.14. s_γ and s_q have the same values.

$$\begin{aligned}q_{ult} &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 22.2 \times 1.42 \times 1.14) + (0.5 \times 18 \times 2 \times 22.5 \times 0.73 \times 1.0) + 18 \times 1 \\ &= 942 + 18 = 960 \text{ kPa}\end{aligned}$$

$$q_a = \frac{q_u}{FS} + \gamma D_f = \frac{942}{3} + 18 \times 1 = 332 \text{ kPa}$$

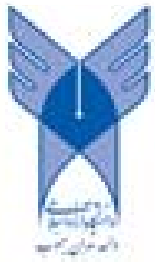
$$R = q_{ult} \times A = 960 \times (2 \times 3) = 5760 \text{ kN}$$

$$P_a = q_a \times A = 332 \times (2 \times 3) = 1992 \text{ kN}$$

Step 6: Check suitability of assumed foundation size.

LRFD: $\phi R = 0.8 \times 5760 = 4608 \text{ kN} > P_{uf} (= 2750 \text{ kN})$; acceptable.

ASD: $P_a (= 1992 \text{ kN}) > P (= 1800 \text{ kN})$; acceptable.



EXAMPLE 12.5 Allowable Bearing Capacity Due to an Inclined Load

Using the footing geometry of Example 12.1, determine q_a for a load inclined at 20° to the vertical along the footing width (see Figure E12.5).

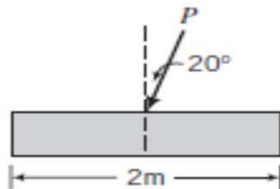


FIGURE E12.5

Strategy You need to use Equation (12.18) for inclined loads. You only need to calculate the inclination factors, since shape and depth factors are not included for load inclination.

Solution 12.5

Step 1: Calculate the inclination factors and depth factors.

$$B' = B; \quad \omega = 20^\circ$$

$$n = n_B = \left(2 + \frac{B'}{L'}\right) / \left(1 + \frac{B'}{L'}\right) = (2 + 1) / (1 + 1) = 1.5$$

$$i_q = \left(1 - \frac{H}{V_n}\right)^n = (1 - \tan \omega)^n = (1 - \tan 20^\circ)^{1.5} = 0.51$$

$$i_y = \left(1 - \frac{H}{V_n}\right)^{n-1} = (1 - \tan \omega)^{n-1} = (1 - \tan 20^\circ)^{1.5-1} = 0.32$$

Step 2: Calculate the ultimate net bearing capacity and allowable bearing capacity.

Use Equation (12.18).

$$q_u = \gamma D_f (N_q - 1) i_q + 0.5 \gamma B N_\gamma i_\gamma = (18 \times 1 \times 32.3 \times 0.51) \\ + (0.5 \times 18 \times 2 \times 37.1 \times 0.32) = 510 \text{ kPa}$$

$$q_a = \frac{q_u}{\text{FS}} + \gamma D_f = \frac{510}{3} + 18 \times 1 = 188 \text{ kPa}$$

The allowable bearing capacity for a vertical centric load is 523 kPa from Example 12.1.

Reduction in allowable bearing capacity is $\frac{523 - 188}{523} = 0.64 = 64\%$.

بارگذاری خارج از مرکز

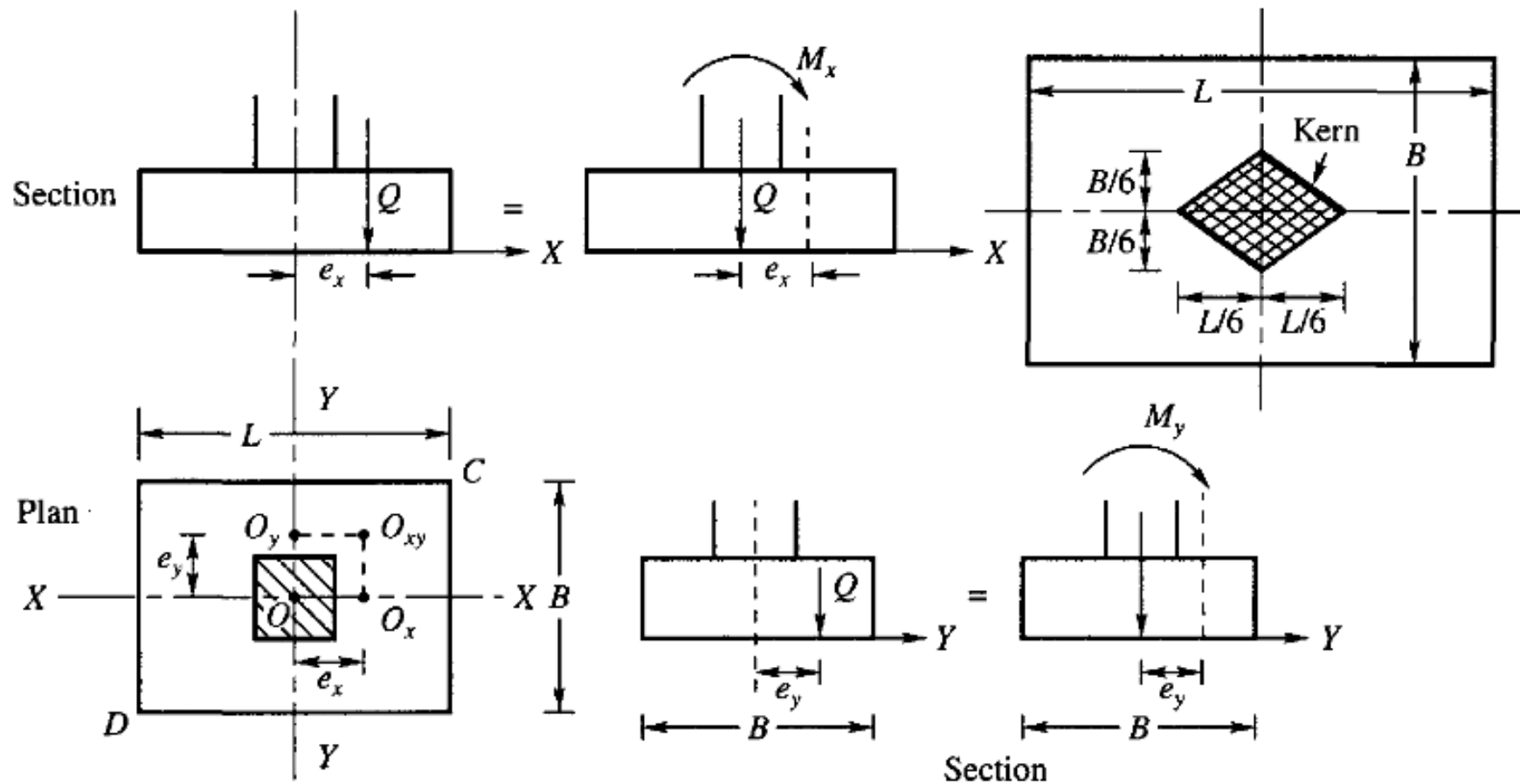
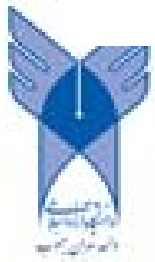


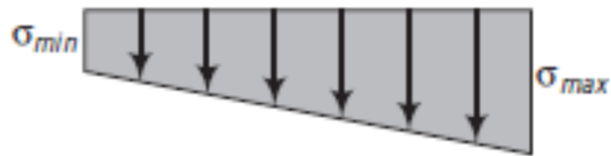
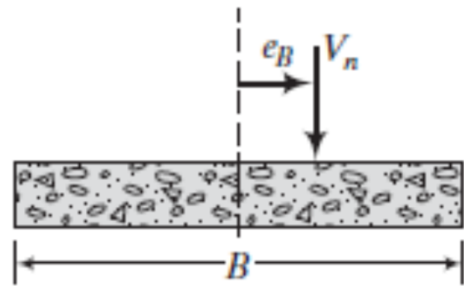
Figure 12.15 Footing subjected to eccentric loadings



بارگذاری خارج از مرکز

Eccentric Loads

$$e_B = \frac{M_y}{V_n}, \quad e_L = \frac{M_x}{V_n}, \quad Z = \frac{I}{y} = \frac{\frac{B^3 L}{12}}{\frac{B}{2}} = \frac{B^2 L}{6} \text{ or } \frac{BL^2}{6}$$



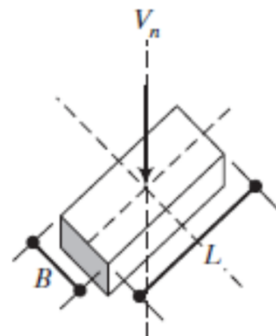
$$\sigma = \frac{V_n}{A} \pm \frac{My}{I} = \frac{V_n}{A} \pm \frac{V_n e y}{I} = \frac{V_n}{A} \pm \frac{V_n e}{Z}$$

$$\sigma_{min} = \frac{V_n}{A} - \frac{V_n e}{Z} = \frac{V_n}{BL} \left(1 - \frac{6e_B}{B} \right)$$

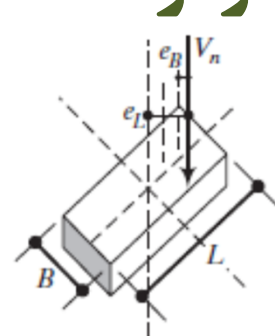
$$\sigma_{max} = \frac{V_n}{A} + \frac{V_n e}{Z} = \frac{V_n}{BL} \left(1 + \frac{6e_B}{B} \right)$$

$$\sigma_{min} = \frac{V_n}{A} - \frac{V_n e}{Z} = \frac{V_n}{BL} \left(1 - \frac{6e_L}{L} \right)$$

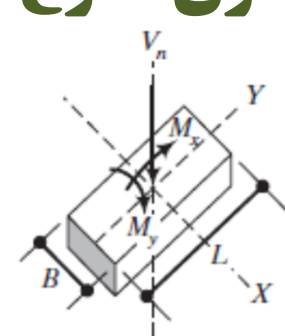
$$\sigma_{max} = \frac{V_n}{A} + \frac{V_n e}{Z} = \frac{V_n}{BL} \left(1 + \frac{6e_L}{L} \right)$$



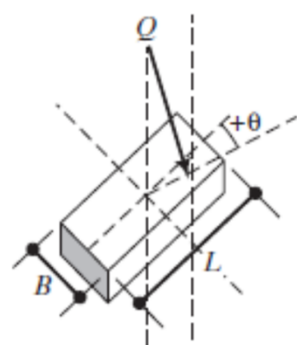
(a) Vertical centric
 $B' = B, L' = L$
 $A' = A = BL$



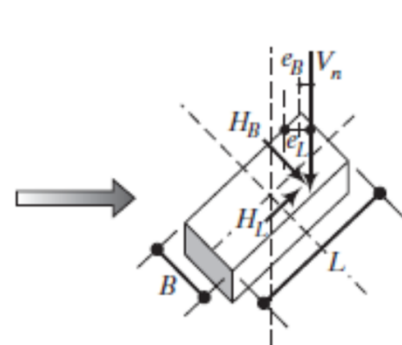
(b) Vertical eccentric
 $B' = B - 2e_B, L' = L - 2e_L$
 $A' = B'L'$



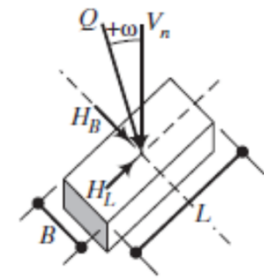
(c) Vertical centric load and moments
 $B' = B, L' = L$
 $B' = B - 2e_B, L' = L - 2e_L$
 $A' = B'L'$



(d) Inclined eccentric load
Set shape and depth factors to 1



(e) Use both the inclination factors and the effective width in the equations
 $B' = B - 2e_B, L' = L - 2e_L$
 $A' = B'L'$



(f) Centric inclined load
 $V_n = Q \cos \omega$
 $H_B = Q \sin \omega; A = BL$
Set shape and depth factors to 1.

If, however, $e_B > \frac{B}{6}$ or $e_L > \frac{L}{6}$

$$P_u = q_u B' L'$$



بار گذاری خارج از مرکز

۱۰-۱۰-۳ توزیع تنش در زیر پی های با خروج از مرکزیت بار

بدست آوردن توزیع تنش زیر پی در هنگام کنترل تنش های حداکثر در خاک، کنترل بلند شدن پی و در زمان طراحی سازه پی بسیار ضروری است. با استفاده از اصول مقاومت مصالح و استاتیک به راحتی می توان حالات زیر را در یک پی تفکیک نمود و توزیع تنش در هر یک از این حالات را محاسبه نمود:

۱- خروج از مرکزیت یک طرفه: در این حالت بار فقط در یکی از دو امتداد خارج از مرکز بوده و در امتداد دیگر خروج از مرکزیت وجود ندارد. توزیع تنش با استفاده از روابط زیر محاسبه می شود.

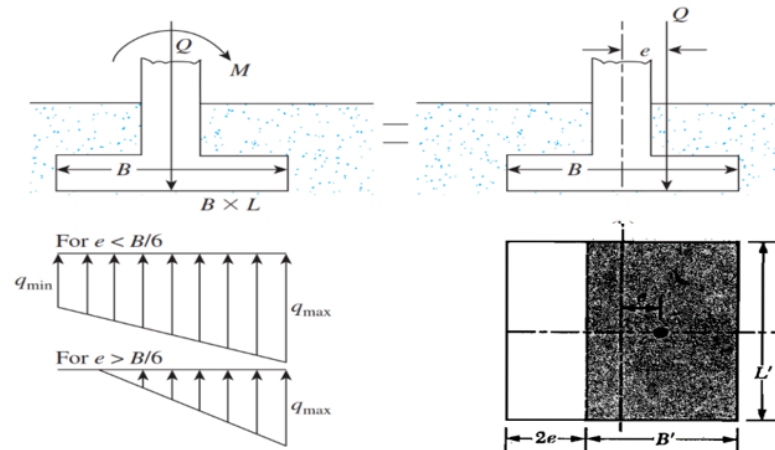
$$e < \frac{B}{6} \text{ (الف): } q_{max,min} = \frac{P}{A} \left(1 \pm \frac{6e}{B}\right) \quad (10-18 \text{ الف})$$

$$e > \frac{B}{6} \text{ (ب): } q_{max} = \frac{4P}{3L(B-2e)} \quad (10-18 \text{ ب})$$

که در آن:

q_{max} و q_{min} = در شکل (۱۰-۲۶) نمایش داده شده است. علامت (+) q_{max} و علامت (-) q_{min} را بدست می دهد.

A = مساحت پی



شکل ۱۰-۲۵ بار خارج از مرکز

تذکره (۱): در حالتی که خروج از مرکزیت پی بیش از $\frac{B}{6}$ باشد ($e > \frac{B}{6}$)، بخشی از پی به کشش می فاند. از آنجایی که خاک قادر به تحمل کشش نمی باشد، از این بخش صرف نظر می شود و با نوشتن معادله تعادل جدید رابطه (۱۰-۱۸) ب) بدست می آید.

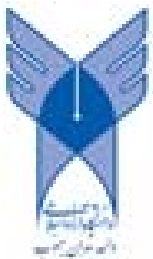
تذکره (۲): هر یک از روابط (۱۰-۱۸) را می توان برای حالتی که خروج از مرکزیت در راستای طول پی (L) اتفاق افتاده باشد، بکار برد. در این حالت کافی است در این روابط، جای B ، L را با هم تعویض نمود.

۲- خروج از مرکزیت دو طرفه: در این حالت خروج از مرکزیت در هر دو امتداد اتفاق می افتد. در

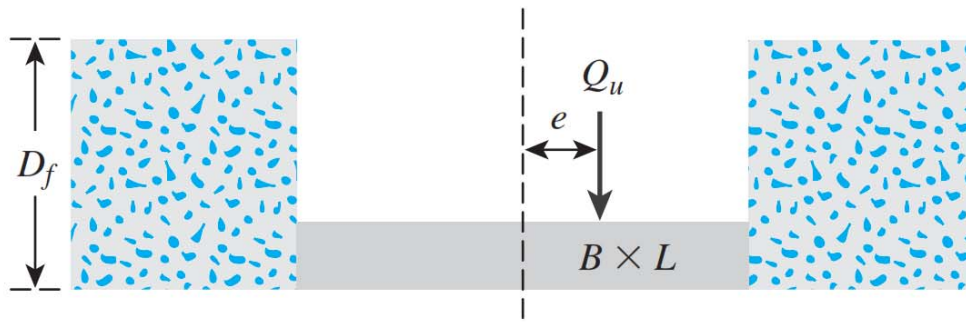
این حالت توزیع تنش از رابطه زیر محاسبه می شود:

$$q = \frac{P}{A} \left(1 \pm \frac{6e_B}{B} \pm \frac{6e_L}{L}\right) \quad (10-19)$$

کلیه پارامترها در این رابطه قبلاً تعریف شده اند.



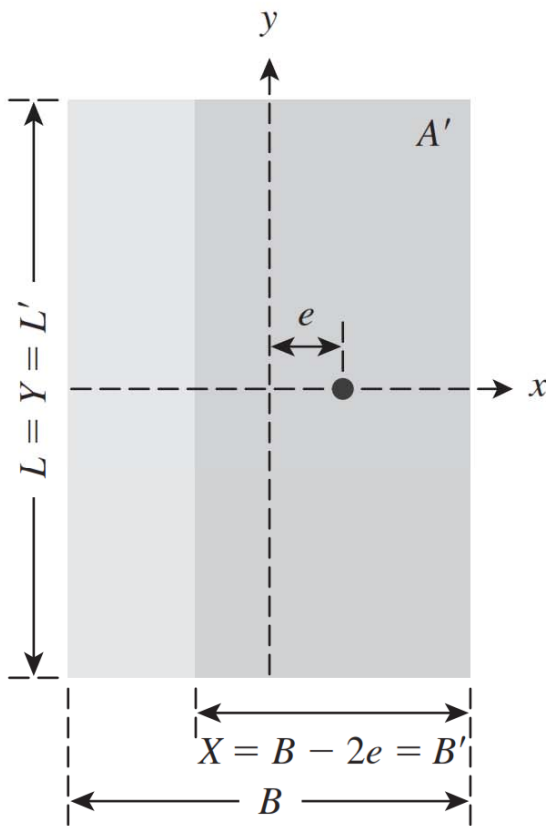
بارگذاری خارج از مرکز



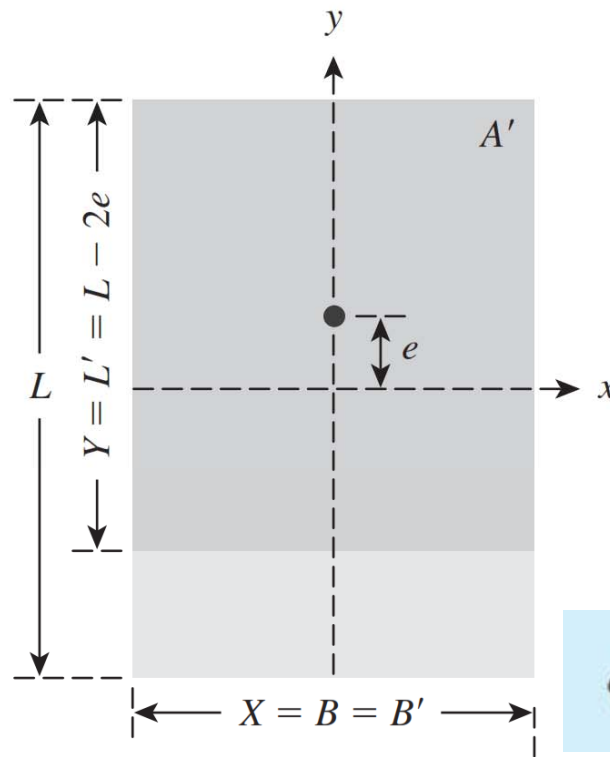
(a) Section

$$X = B - 2e$$

$$Y = L - 2e$$



(b) Plan



(c) Plan

$$B' = X \text{ or } Y, \text{ whichever is smaller}$$

$$L' = X \text{ or } Y, \text{ whichever is larger}$$

$$q_u = c' \lambda_{cs} \lambda_{cd} N_c + q \lambda_{qs} \lambda_{qd} N_q + \frac{1}{2} \lambda_{\gamma s} \lambda_{\gamma d} \gamma B' N_\gamma$$

$$Q_u = q_u (B' L') = q_u A'$$



بارگذاری خارج از مرکز

۱۰-۱۰-۱ استفاده از ضرایب اصلاح مایرهورف

مایرهورف پیشنهاد می کند که برای تعیین ظرفیت باربری یک پی خارج از مرکز از روش زیر استفاده شود:

- ۱- ابتدا ظرفیت باربری پی بدون در نظر گرفتن خروج از مرکزیت محاسبه می شود.
- ۲- ظرفیت باربری با در نظر گرفتن خروج از مرکزیت از رابطه زیر محاسبه می شود.

$$q'_{ult} = q_{ult} R_B R_L \quad (10-15)$$

که در آن

q'_{ult} و q_{ult} = به ترتیب ظرفیت باربری با و بدون در نظر گرفتن خروج از مرکزیت
 R_B و R_L = ضرایب اصلاح مایرهورف که از رابطه های زیر محاسبه می شود

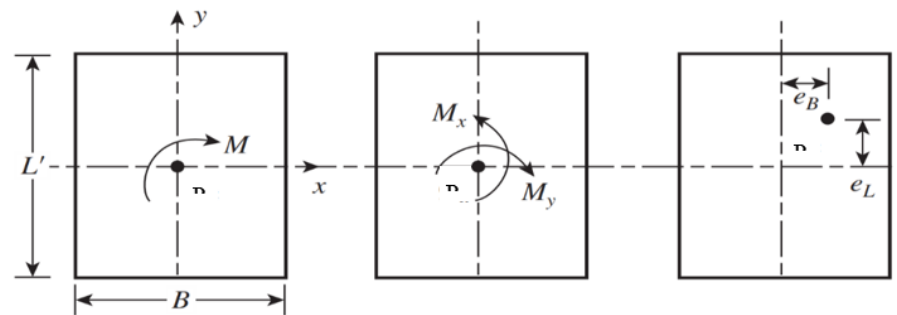
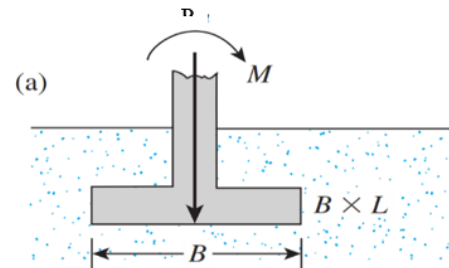
$$R_B = \left(1 - \frac{2e_B}{B}\right) \quad \text{در خاک های چسبنده:}$$

$$R_L = \left(1 - \frac{2e_L}{L}\right)$$

$$R_B = 1 - \sqrt{\frac{e_B}{B}} \quad \text{در خاک های غیرچسبنده:}$$

$$R_L = 1 - \sqrt{\frac{e_L}{L}}$$

مقادیر e_B و e_L در شکل (۱۰-۲۲) تعریف شده اند.



Purkayastha and Char (1977) carried out stability analysis of eccentrically loaded *continuous foundations on granular soil* (i.e., $c' = 0$) using the method of slices. Based on that analysis, they proposed that

$$R_k = 1 - \frac{q_{u(\text{eccentric})}}{q_{u(\text{centric})}} \quad (16.43)$$

where R_k = reduction factor

$q_{u(\text{eccentric})}$ = ultimate bearing capacity of eccentrically loaded continuous foundations

$$= \frac{Q_{u(\text{eccentric})}}{B}$$

$q_{u(\text{centric})}$ = ultimate bearing capacity of centrally loaded continuous foundations

$$= \frac{Q_{u(\text{centric})}}{B}$$

The magnitude of R_k can be expressed as

$$R_k = a \left(\frac{e}{B} \right)^k \quad (16.44)$$

where a and k are functions of the embedment ratio D_f/B (Table 16.6).

Hence, combining Eqs. (16.43) and (16.44) gives

$$Q_{u(\text{eccentric})} = Q_{u(\text{centric})} \left[1 - a \left(\frac{e}{B} \right)^k \right] \quad (16.45)$$

where $Q_{u(\text{eccentric})}$ and $Q_{u(\text{centric})}$ = ultimate load per unit length, respectively, for eccentrically and centrally loaded foundations.

Table 16.6 Variations of a and k [Eq. (16.44)]

D_f/B	a	k
0	1.862	0.73
0.25	1.811	0.785
0.5	1.754	0.80
1.0	1.820	0.888



Example 12.16

Figure Ex. 12.16 gives the plan of a footing subjected to eccentric load with two way eccentricity. The footing is founded at a depth 3 m below the ground surface. Given $e_x = 0.60$ m and $e_y = 0.75$ m, determine Q_{ult} . The soil properties are: $c = 0$, $N_{cor} = 20$, $\gamma = 18.5$ kN/m³. The soil is medium dense sand. Use N_γ (Meyerhof) from Table 12.2 and Hansen's shape and depth factors from Table 12.3.

Solution

Figure Ex. 12.16 shows the two-way eccentricity. The effective lengths and breadths of the foundation from Eq. (12.36a) is

$$B' = B - 2e_y = 6 - 2 \times 0.75 = 4.5 \text{ m.}$$

$$L' = L - 2e_x = 6 - 2 \times 0.6 = 4.8 \text{ m.}$$

$$\text{Effective area, } A' = L' \times B' = 4.5 \times 4.8 = 21.6 \text{ m}^2$$

As in Example 12.15

$$q'_u = \gamma D_f N_q s_q d_q + \frac{1}{2} \gamma B' N_\gamma s_\gamma d_\gamma$$

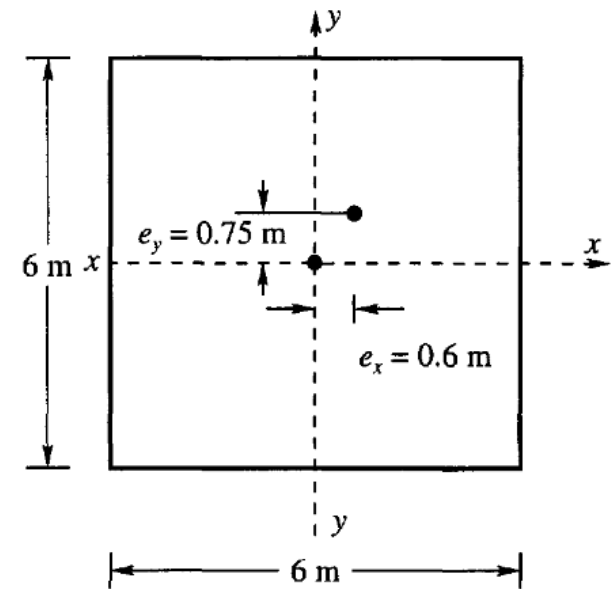


Figure Ex. 12.16



For $\phi = 33^\circ$, $N_q = 26.3$ and $N_\gamma = 26.55$ (Meyerhof)

From Table 12.3 (Hansen)

$$s_q = 1 + \frac{B'}{L'} \tan 33^\circ = 1 + \frac{4.5}{4.8} \times 0.65 = 1.61$$

$$s_\gamma = 1 - 0.4 \frac{B'}{L'} = 1 - 0.4 \times \frac{4.5}{4.8} = 0.63$$

$$d_q = 1 + 2 \tan 33^\circ (1 - \sin 33^\circ)^2 \times \frac{3}{4.5}$$
$$= 1 + 1.3 \times 0.21 \times 0.67 = 1.183$$

$$d_\gamma = 1$$

Substituting

$$q'_u = 18.5 \times 3 \times 26.3 \times 1.61 \times 1.183 + \frac{1}{2} \times 18.5 \times 4.5 \times 26.55 \times 0.63 \times (1)$$
$$= 2,780 + 696 = 3,476 \text{ kN/m}^2$$

$$Q_{ult} = A' q'_u = 21.6 \times 3,476 = 75,082 \text{ kN}$$

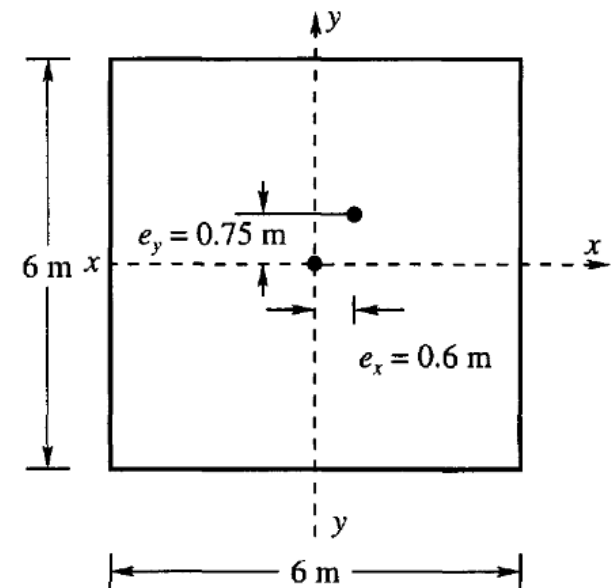


Figure Ex. 12.16



Bearing Capacity of Sand Based on Settlement

Obtaining undisturbed specimens of cohesionless sand during a soil exploration program is usually difficult. For this reason, the results of standard penetration tests (SPTs) performed during subsurface exploration are commonly used to predict the allowable soil-bearing capacity of foundations on sand. (The procedure for conducting SPTs is discussed in detail in Chapter 18.)

Meyerhof (1956) proposed correlations for the *net allowable bearing capacity* (q_{net}) based on settlement (elastic). It was further revised by Meyerhof (1965) based on the field performance of foundations. The correlations can be expressed as follows.

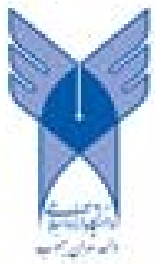
SI Units

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.05} F_d \left[\frac{S_e(\text{mm})}{25} \right] \quad (\text{for } B \leq 1.22 \text{ m}) \quad (16.46)$$

$$q_{net}(\text{kN/m}^2) = \frac{N_{60}}{0.08} \left(\frac{B + 0.3}{B} \right)^2 F_d \left[\frac{S_e(\text{mm})}{25} \right] \quad (\text{for } B > 1.22 \text{ m}) \quad (16.47)$$

where B = foundation width (m)

S_e = settlement



13.5 EMPIRICAL EQUATIONS BASED ON SPT VALUES FOR FOOTINGS ON COHESIONLESS SOILS

Footings on granular soils are sometimes proportioned using empirical relationships. Teng (1969) proposed an equation for a settlement of 25 mm based on the curves developed by Terzaghi and Peck (1948). The modified form of the equation is

$$q_s = 35(N_{cor} - 3) \frac{B + 0.3}{2B}^2 R_{w2} F_d \text{ kN/m}^2 \quad (13.15a)$$

where q_s = net allowable bearing pressure for a settlement of 25 mm in kN/m^2 ,

N_{cor} = corrected standard penetration value

R_{w2} = water table correction factor (Refer Section 12.7)

F_d = depth factor = $(1 + D_f / B) \leq 2.0$

B = width of footing in meters,

D_f = depth of foundation in meters.

Meyerhof (1956) proposed the following equations which are slightly different from that of Teng

$$q_s = 12 N_{cor} R_{w2} F_d \quad \text{for } B \leq 1.2 \text{ m} \quad (13.15b)$$

$$q_s = 8 N_{cor} \frac{B + 0.3}{B}^2 R_{w2} F_d \quad \text{for } B > 1.2 \text{ m} \quad (13.15c)$$

where $F_d = (1 + 0.33 D_f / B) \leq 1.33$.

Experimental results indicate that the equations presented by Teng and Meyerhof are too conservative. Bowles (1996) proposes an approximate increase of 50 percent over that of Meyerhof which can also be applied to Teng's equations. Modified equations of Teng and Meyerhof are,

Teng's equation (modified),

$$q_s = 53 (N_{cor} - 3) \frac{B + 0.3}{2B}^2 R_{w2} F_d \quad (13.16a)$$

Meyerhof's equation (modified)

$$q_s = 20 N_{cor} R_{w2} F_d \text{ for } B \leq 1.2 \text{ m} \quad (13.16b)$$

$$q_s = 12.5 N_{cor} \frac{B + 0.3}{B}^2 R_{w2} F_d \text{ for } B > 1.2 \text{ m} \quad (13.16c)$$

If the tolerable settlement is greater than 25 mm, the safe bearing pressure computed by the above equations can be increased linearly as,

$$q'_s = \frac{S'}{25} q_s \quad (13.16d)$$

where q'_s = net safe bearing pressure for a settlement S' mm, q_s = net safe bearing pressure for a settlement of 25 mm.



13.6 SAFE BEARING PRESSURE FROM EMPIRICAL EQUATIONS BASED ON CPT VALUES FOR FOOTINGS ON COHESIONLESS SOIL

The static cone penetration test in which a standard cone of 10 cm² sectional area is pushed into the soil without the necessity of boring provides a much more accurate and detailed variation in the soil as discussed in Chapter 9. Meyerhof (1956) suggested a set of empirical equations based on the Terzaghi and Peck curves (1948). As these equations were also found to be conservative, modified forms with an increase of 50 percent over the original values are given below.

$$q_s = 3.6q_c R_{w2} \text{ kPa} \quad \text{for } B \leq 1.2 \text{ m} \quad (13.17a)$$

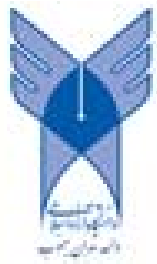
$$q_s = 2.1q_c \left(1 + \frac{1}{B}\right)^2 R_{w2} \text{ kPa} \quad \text{for } B > 1.2 \text{ m} \quad (13.17b)$$

An approximate formula for all widths

$$q_s = 2.7q_c R_{w2} \text{ kPa} \quad (13.17c)$$

where q_c is the cone point resistance in kg/cm² and q_s in kPa.

The above equations have been developed for a settlement of 25 mm.



ظرفیت باربری با استفاده از بار گذاری صفحه

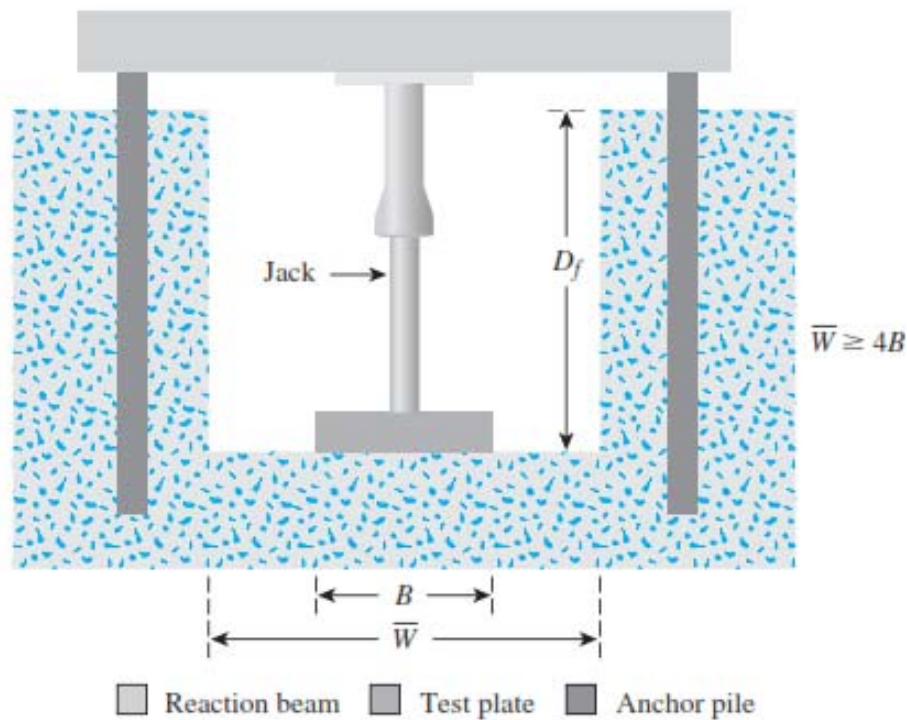


Figure 16.16 Diagram of plate load test

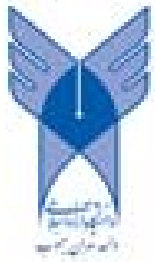
From the results of field load tests, the ultimate soil-bearing capacity of actual footings can be approximated as follows:

For clays,

$$q_{u(\text{footing})} = q_{u(\text{plate})} \quad (16.53)$$

For sandy soils,

$$q_{u(\text{footing})} = q_{u(\text{plate})} \frac{B_{(\text{footing})}}{B_{(\text{plate})}} \quad (16.54)$$



For a given intensity of load q , the settlement of the actual footing also can be approximated from the following equations:

In clay,

$$S_{e(\text{footing})} = S_{e(\text{plate})} \frac{B_{(\text{footing})}}{B_{(\text{plate})}} \quad (16.55)$$

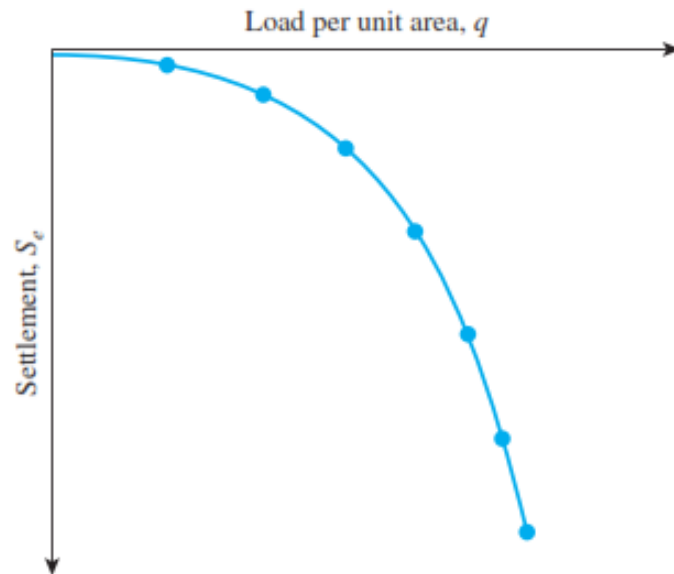
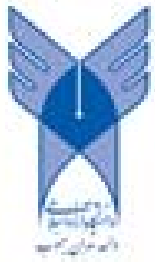


Figure 16.17 Typical load-settlement curve obtained from plate load test



For clays,

$$q_{u(\text{footing})} = q_{u(\text{plate})} \quad (15.48)$$

For sandy soils,

$$q_{u(\text{footing})} = q_{u(\text{plate})} \frac{B_{(\text{footing})}}{B_{(\text{plate})}} \quad (15.49)$$

For a given intensity of load q , the settlement of the actual footing can also be approximated from the following equations:

In clay,

$$s_{e(\text{footing})} = s_{e(\text{plate})} \frac{B_{(\text{footing})}}{B_{(\text{plate})}} \quad (15.50)$$

In sandy soil,

$$s_{e(\text{footing})} = s_{e(\text{plate})} \left[\frac{2B_{(\text{footing})}}{B_{(\text{footing})} + B_{(\text{plate})}} \right]^2 \quad (15.51)$$



۱۱-۱۰ اثر ناهمگنی خاک

یکی از فرضیات روش های تعیین ظرفیت باربری همگن بودن خاک است، در حالیکه در عمل به ندرت می توان این فرض را درباره یک خاک معتبر دانست.

نکته: در یک خاک که از چند لایه مختلف با مقادیر چسبندگی c_1 و زاویه اصطکاک ϕ_1 و ضخامت H_1 تشکیل شده باشد می توان مقادیر چسبندگی و زاویه اصطکاک معادل را از روابط زیر تعیین کرد:

$$c_{eq} = \frac{\sum_{i=1}^n c_i H_i}{\sum_{i=1}^n H_i} \quad (الف-۱۰-۲۰)$$

$$\phi_{eq} = \text{Arctan} \left(\frac{\sum_{i=1}^n H_i \tan \phi_i}{\sum_{i=1}^n H_i} \right) \quad (ب-۱۰-۲۰)$$

که در آن:

c_i و ϕ_i = به ترتیب چسبندگی و زاویه اصطکاک لایه i ام
 H_i = ضخامت لایه i ام

c_{eq} و ϕ_{eq} = به ترتیب چسبندگی و زاویه اصطکاک خاک معادل

تذکر: روابط فوق در واقع یک نوع میانگین گیری وزنی است. در این روش فقط لایه های خاک تا عمق $H = 0.5 B \tan(45 + \frac{\phi}{2})$ در نظر گرفته می شوند. بنابراین این روش منجر به یک سعی و خطا خواهد شد. به مثال بعد دقت کنید.

مثال: برای یک پی مطابق شکل مطلوب است محاسبه ظرفیت باربری به روش ترزاقی.

پاسخ: قبل از محاسبه ظرفیت می بایست ابتدا مقدار c و ϕ را در این خاک تعیین کنیم. برای اینکار با استفاده از روش پیش گفته، ابتدا مقدار $\phi_{eq} = 28^\circ$ حدس زده می شود. با استفاده از رابطه (۱۰-۲۰-ب) داریم:

$$H = 0.5 B \tan \left(45 + \frac{\phi}{2} \right) = (0.5)(2) \tan \left(45 + \frac{30}{2} \right) = 1.73 \text{ m}$$

$$\phi_{eq} = \text{Arctan} \left(\frac{\sum_{i=1}^n H_i \tan \phi_i}{\sum_{i=1}^n H_i} \right) \\ = \text{Arctan} \left(\frac{0.5 \times \tan 35 + 0.19 \tan 21}{0.69} \right) = 31.5^\circ \approx 30^\circ$$

با توجه به اینکه مقادیر ϕ_{eq} فرضی و محاسبه شده تقریباً برابر هستند، عملیات سعی و خطا را متوقف کرده و $\phi_{eq} = 31^\circ$ انتخاب می گردد. با استفاده از رابطه (۱۰-۲۰-الف) داریم:

$$c_{eq} = \frac{\sum c_i H_i}{\sum H_i} = \frac{(0.2)(0.5) + (0.8)(0.19)}{0.69} = 0.37 \text{ ton/m}^2$$

همچنین مقدار γ_{eq} محاسبه می شود:

$$\gamma_{eq} = \frac{\sum \gamma_i H_i}{\sum H_i} = \frac{(1.7)(0.5) + (1.5)(0.19)}{0.69} = 1.65 \text{ ton/m}^3$$

با استفاده از جدول (۱۰-۱) داریم:

$$\phi_{eq} = 31^\circ \Rightarrow N_c = 40.41; \quad N_q = 25.28; \quad N_\gamma = 22.65$$

با توجه به مربعی بودن پی داریم:

$$S_c = 1.3; \quad S_\gamma = 0.8$$

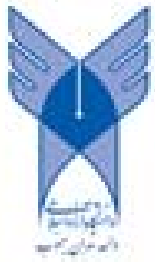
با نوشتن رابطه ترزاقی داریم:

$$q_{ult} = c N_c S_c + \bar{q} N_q S_q + 0.5 \gamma B N_\gamma S_\gamma \\ q_{ult} = (0.37)(40.41)(1.3) + (1.8 \times 1)(25.28) + 0.5(1.65)(2)(22.65)(0.8) \\ q_{ult} = 95 \text{ ton/m}^2$$



تحليل پي بر اساس سرعت بارگذاري

- ۱- نقش سرعت بارگذاري
- ۲- ارتباط زهكشي و سرعت
- ۳- آناليز کوتاه مدت و دراز مدت



General Loading Conditions

TSA: $q_u = 5.14 s_u s_c d_c i_c b_c g_c$

ESA: $q_u = \gamma D_f (N_q - 1) s_q d_q i_q b_q g_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma b_\gamma g_\gamma$

Ultimate Gross Bearing Capacity: $q_{ult} = q_u + \gamma D_f$

Allowable Bearing Capacity: $q_a = \frac{q_u}{FS} + \gamma D_f$

$$N_q = e^{\pi \tan \phi'_p} \tan^2 \left(45^\circ + \frac{\phi'_p}{2} \right)$$

Vesic (1973): $N_\gamma = 2(N_q + 1) \tan \phi'_{ps}$

Meyerhof (1976): $N_\gamma = (N_q - 1) \tan(1.4\phi'_p)$

$$(\phi'_p)_{ps} = \frac{9}{8} (\phi'_p)_{tr}$$

Vertical Centric Load Only on a Horizontal Footing Resting on a Horizontal Surface

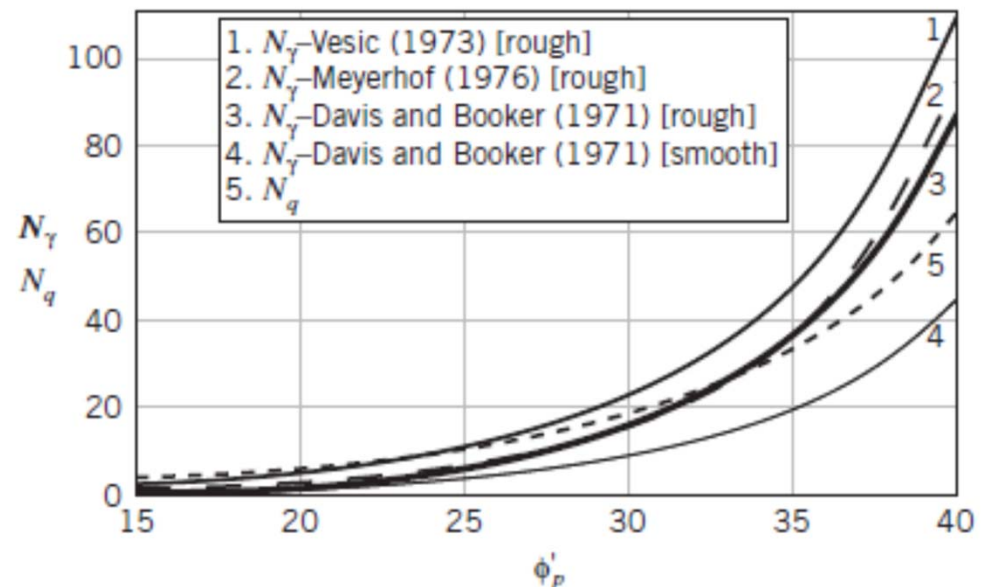
TSA: $q_u = 5.14 s_u s_c d_c$

ESA: $q_u = \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma$

Inclined Load Only on a Horizontal Footing

TSA: $q_u = 5.14 s_u i_c$

ESA: $q_u = \gamma D_f (N_q - 1) i_q + 0.5 \gamma B' N_\gamma i_\gamma$



Geometric Factors for Use in Theoretical Bearing Capacity Equations

Geometric parameters for TSA

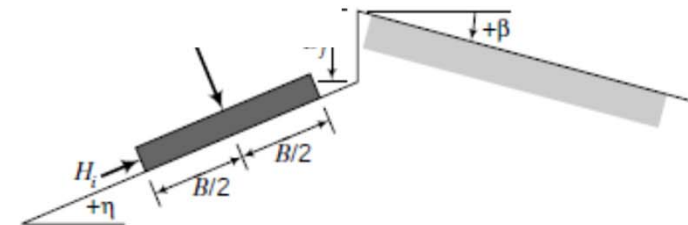
s_c	d_c	i_c	b_c	g_c
$1 + 0.2 \frac{B'}{L'}$	$1 + 0.33 \tan^{-1} \frac{D_f}{B'}$ See note 1	$1 - \frac{nH}{5.14 s_u B' L'}$ See note 2	$1 - \frac{\eta^\circ}{147}$ $\beta < \phi'_p; \eta^\circ + \beta^\circ < 90^\circ$ See Figure 12.9	$1 - \frac{\beta^\circ}{147}$ $\beta < \phi'_p; \eta^\circ + \beta^\circ < 90^\circ$ See Figure 12.9

Geometric parameters for ESA

s_q	d_q	i_q	b_q	g_q
$1 + \frac{B'}{L'} \tan \phi'_p$	$1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \left(\frac{D_f}{B'} \right)$	$\left(1 - \frac{H}{V_n} \right)^n$ See note 2	$(1 - \eta \tan \phi'_p)^2$ η is in radians	$(1 - \tan \beta)^2$
s_γ	d_γ	i_γ	b_γ	g_γ
$1 - 0.4 \frac{B'}{L'}$	1	$\left(1 - \frac{H}{V_n} \right)^{n+1}$ See note 2	$b_\gamma = b_q$	$g_\gamma = g_q$

Note 1: If the shear strength of the soil above the footing is low compared with that of the soil below the footing, you should set all depth factors to 1. The term $\tan^{-1} \left(\frac{D_f}{B'} \right)$ is in radians.

Note 2: The depth and shape factors for inclined loads should be set to 1. For loading inclined in the direction of the width, B , $\theta = 90^\circ$ in Figure 12.10d, $n = n_B = \left(2 + \frac{B'}{L'} \right) / \left(1 + \frac{B'}{L'} \right)$. For loading inclined in the direction of the length, L , $\theta = 0^\circ$ in Figure 12.10d, $n = n_L = \left(2 + \frac{L'}{B'} \right) / \left(1 + \frac{L'}{B'} \right)$. For other loading conditions, $n = n_L \cos^2 \theta + n_B \sin^2 \theta$.

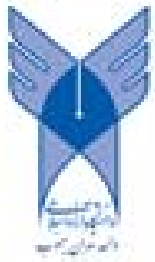


The General Bearing Capacity Eqn.

Considers

- rigidity of soil, (r)
- shape of footing, (s)
- depth of embedment, (d)
- Inclined load, (i)
- base inclination, (b)
- ground inclination, (g)

$$q_u = r_c s_c d_c i_c b_c g_c (c' N_c) + r_q s_q d_q i_q b_q g_q (q_o N_q) + r_\gamma s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma (0.5 \gamma' B N_\gamma)$$



Note: shape factors not used with inclination factors

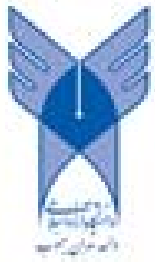
نشست پی

Components of Total Settlement

The total settlement of a foundation comprises three parts as follows

$$S = S_e + S_c + S_s \quad (13.18)$$

where S = total settlement
 S_e = elastic or immediate settlement
 S_c = consolidation settlement
 S_s = secondary settlement



۱-۱۳-۱ نشست های مجاز

نشست های یک سازه دو دیگانه قابل توجه است.

- ۱- نشست کلی
- ۲- نشست نامتقارن (نسبی)
- ۳- در یک سازه عادی نشست های کلی که در همه نقاط اتفاق می افتند، معمولاً آثار منفی اندکی بر رفتار سازه یا تلاشهای ایجاد شده در آن دارند. اما نشست نسبی یک جزء نسبت به جزء دیگر اغلب باعث تبعات منفی شدیدتری خواهد بود.

نکته: در کارهای معمولی از نشست های مجاز زیر استفاده نمائید.

۱- نشست کلی مجاز پی های تکی: 2.5 cm

۲- نشست کلی مجاز پی های گسترده: 5 cm

۳- نشست نسبی مجاز پی های تکی و گسترده: 2 cm

۴- گاهی اوقات برای تعیین میزان نشست مجاز پی های تکی از روش شیب استفاده می شود. مثلاً

اگر شیب مجاز برابر $\frac{1}{500}$ انتخاب شود و فاصله دو ستون برابر با 15 m باشد، حداکثر نشست

نسبی قابل قبول برابر است با:

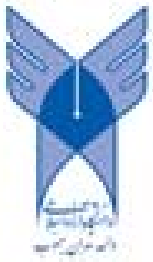
$$\frac{1}{500} \times 10 = 0.02m = 2cm$$

در جدول (۱۰-۵) مقادیر مجاز توصیه شده توسط بیروم ارائه شده است. این مقادیر توسط آئین نامه های گوناگون مورد استفاده قرار گرفته شده است.

جدول ۱۰-۶۱۰ مقادیر حدی مجاز نشست و اعوجاج زاویه ای

Construction and/or material	Maximum δ/L
Masonry (center sag)	1/250-1/700
(edge sag)	1/500-1/1000
Masonry and steel	1/500
Steel with metal siding	1/250
Tall structures	< 1/300 (so tilt not noticeable)
Storage tanks (center-to-edge)	< 1/300

Structure	On sand or hard clay	On plastic clay	Average max. settlement, mm
Crane runway	0.003	0.003	
Steel and concrete frames	0.002	0.002	100
End rows of brick-clad frame	0.0007	0.001	150
Where strain does not occur	0.005	0.005	
Multistory brick wall			25 $L/H \geq 2.5$
L/H to 3	0.0003	0.0004	100 $L/H \leq 1.5$
Multistory brick wall			
L/H over 5	0.0005	0.0007	
One-story mill buildings	0.001	0.001	
Smokestacks, water towers, ring foundations	0.004	0.004	300
Structures on permafrost			
Reinforced concrete	0.002-0.0015		150 at 40 mm/year†
Masonry, precast concrete	0.003-0.002		200 at 60 mm/year
Steel frames	0.004-0.0025		250 at 80 mm/year
Timber	0.007-0.005		400 at 129 mm/year



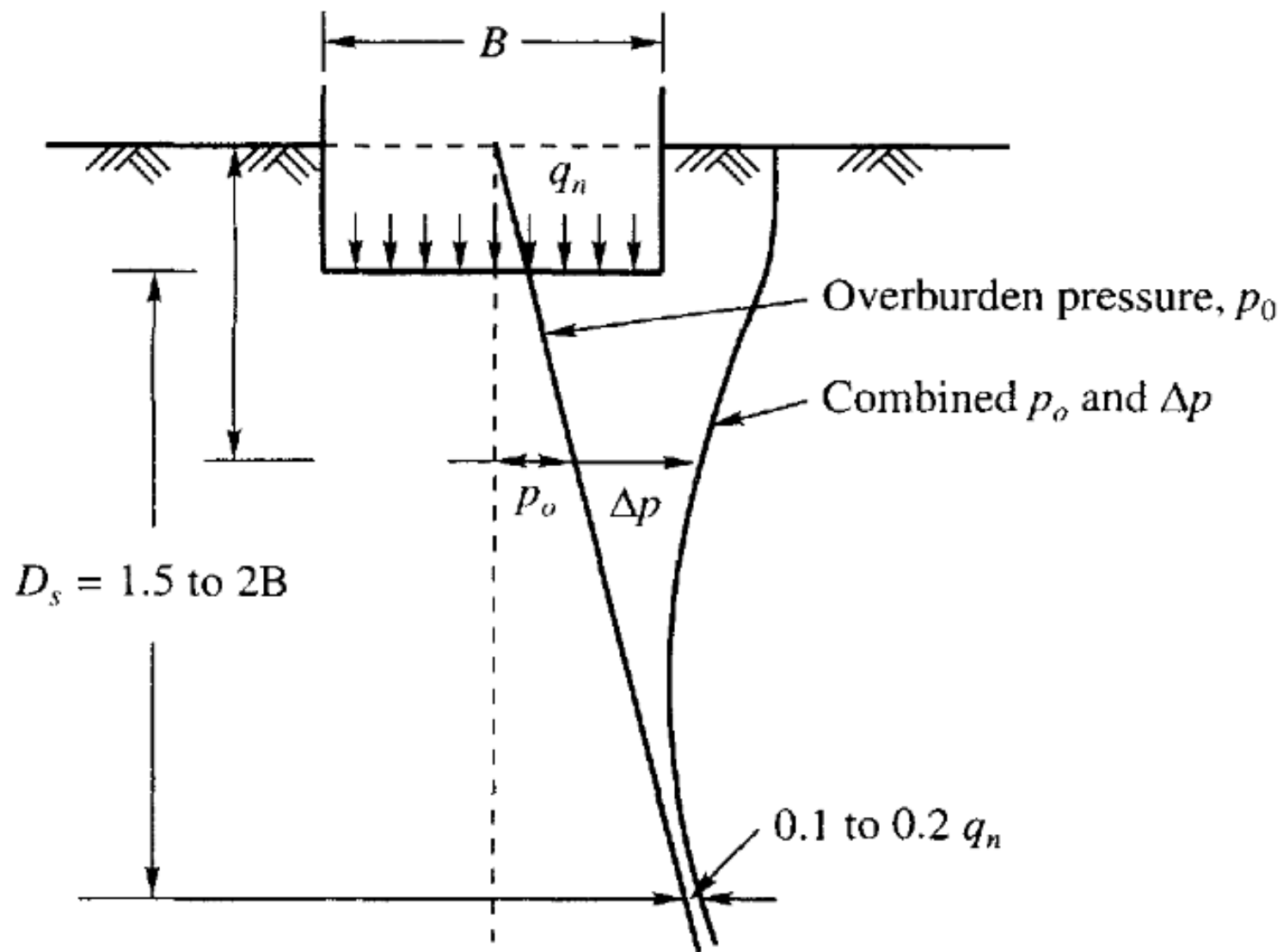


Figure 13.6 Overburden pressure and vertical stress distribution

13.10 ELASTIC SETTLEMENT BENEATH THE CORNER OF A UNIFORMLY LOADED FLEXIBLE AREA BASED ON THE THEORY OF ELASTICITY

The net elastic settlement equation for a flexible surface footing may be written as,

$$S_e = q_n B \frac{(1 - \mu^2)}{E_s} I_f \quad (13.20a)$$

where S_e = elastic settlement
 B = width of foundation,
 E_s = modulus of elasticity of soil,
 μ = Poisson's ratio,
 q_n = net foundation pressure,
 I_f = influence factor.

for Poisson's ratio of 0.5, $I_f = F_1$,

for Poisson's ratio of zero, $I_f = F_1 + F_2$.

where F_1 and F_2 are factors which depend upon the ratios of H/B and L/B .

For intermediate values of μ , the value of I_f can be computed by means of interpolation or by the equation

$$I_f = \left[F_1 + \frac{(1 - \mu - 2\mu^2)F_2}{1 - \mu^2} \right] \quad (13.20b)$$

Table 13.2 Equations for computing E_s by making use of SPT and CPT values (in kPa)

Soil	SPT	CPT
Sand (normally consolidated)	500 ($N_{cor} + 15$) (35000 to 50000) $\log N_{cor}$ (U.S.S.R Practice)	2 to 4 q_c (1 + D_r^2) q_c
Sand (saturated)	250 ($N_{cor} + 15$)	
Sand (overconsolidated)	–	6 to 30 q_c
Gravelly sand and gravel	1200 ($N_{cor} + 6$)	
Clayey sand	320 ($N_{cor} + 15$)	3 to 6 q_c
Silty sand	300 ($N_{cor} + 6$)	1 to 2 q_c
Soft clay	–	3 to 8 q_c

Table 13.3 Typical range of values for Poisson's ratio (Bowles, 1996)

Type of soil	μ
Clay, saturated	0.4–0.5
Clay, unsaturated	0.1–0.3
Sandy clay	0.2–0.3
Silt	0.3–0.35
Sand (dense)	0.2–0.4
Coarse (void ratio 0.4 to 0.7)	0.15
Fine grained (void ratio = 0.4 to 0.7)	0.25
Rock	0.1–0.4

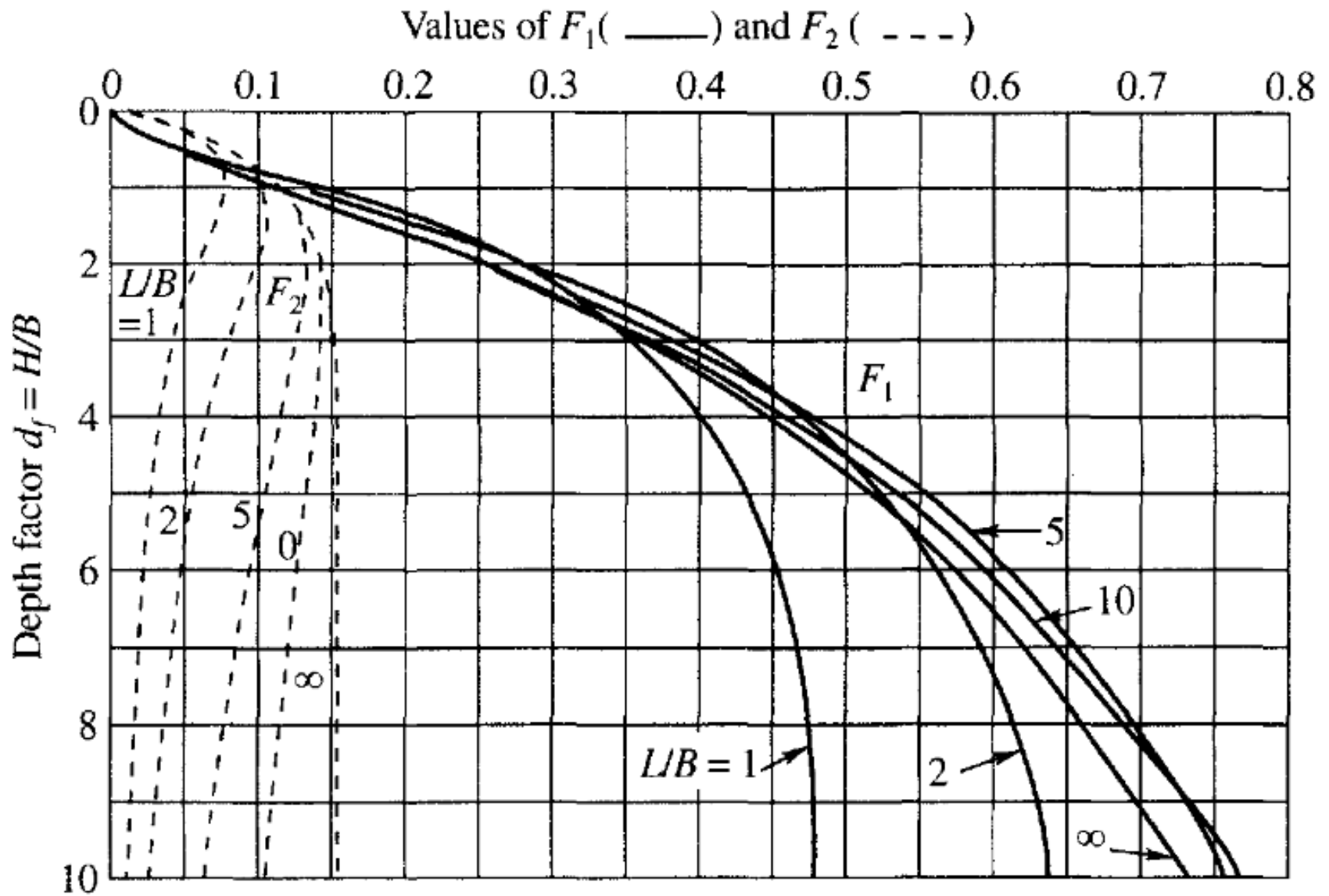
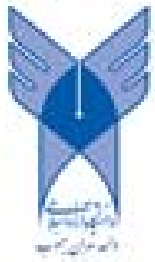
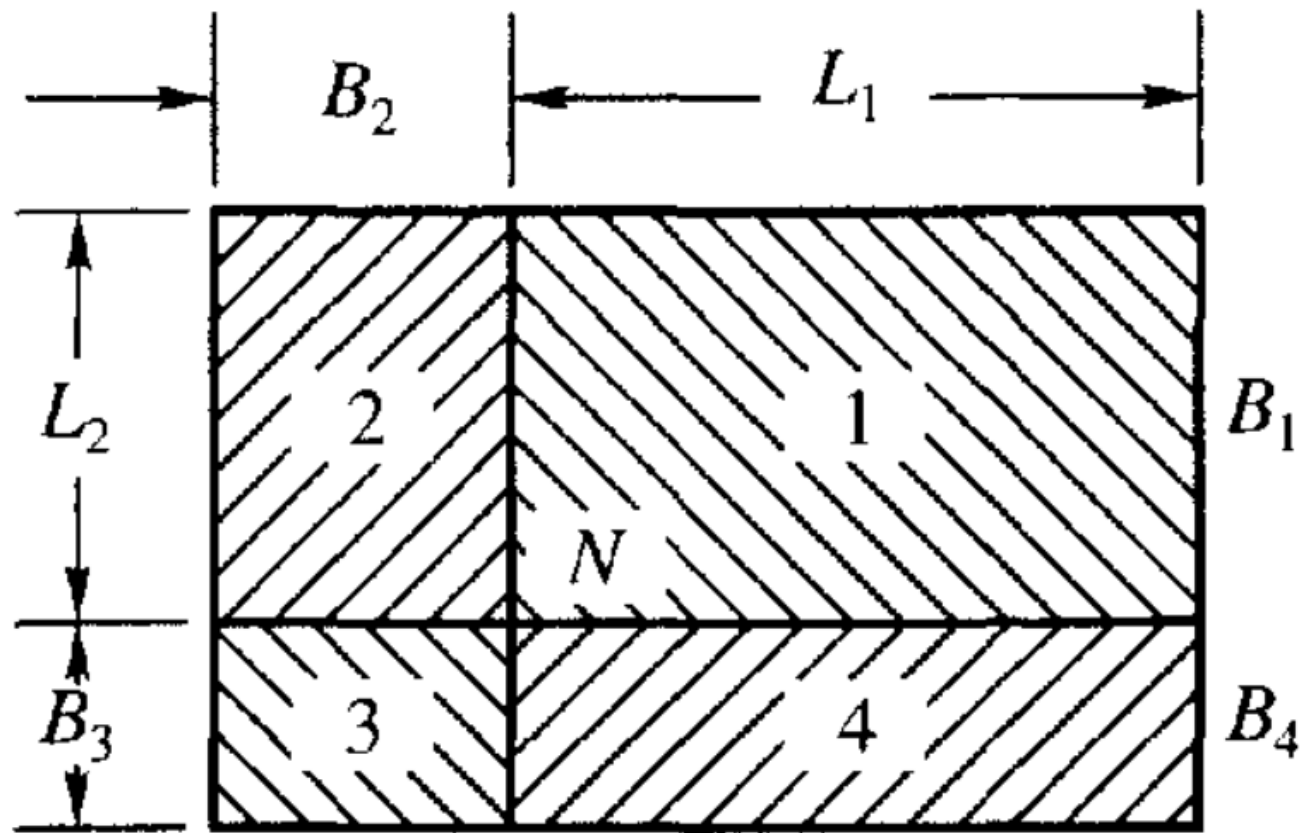


Table 13.4 Influence factor I_f (Bowles, 1988)

Shape	I_f (average values)	
	Flexible footing	Rigid footing
Circle	0.85	0.88
Square	0.95	0.82
Rectangle	1.20	1.06
L/B = 1.5	1.20	1.06
2.0	1.31	1.20
5.0	1.83	1.70
10.0	2.25	2.10
100.0	2.96	3.40





(b)

$$S_e \text{ at point } N = \frac{q_n(1-\mu^2)}{E_s} [I_{f1}B_1 + I_{f2}B_2 + I_{f3}B_3 + I_{f4}B_4]$$

(13.20c)



$$\text{Depth factor} = \frac{\text{Corrected settlement for foundation of depth } D_f}{\text{Calculated settlement for foundation at surface}}$$

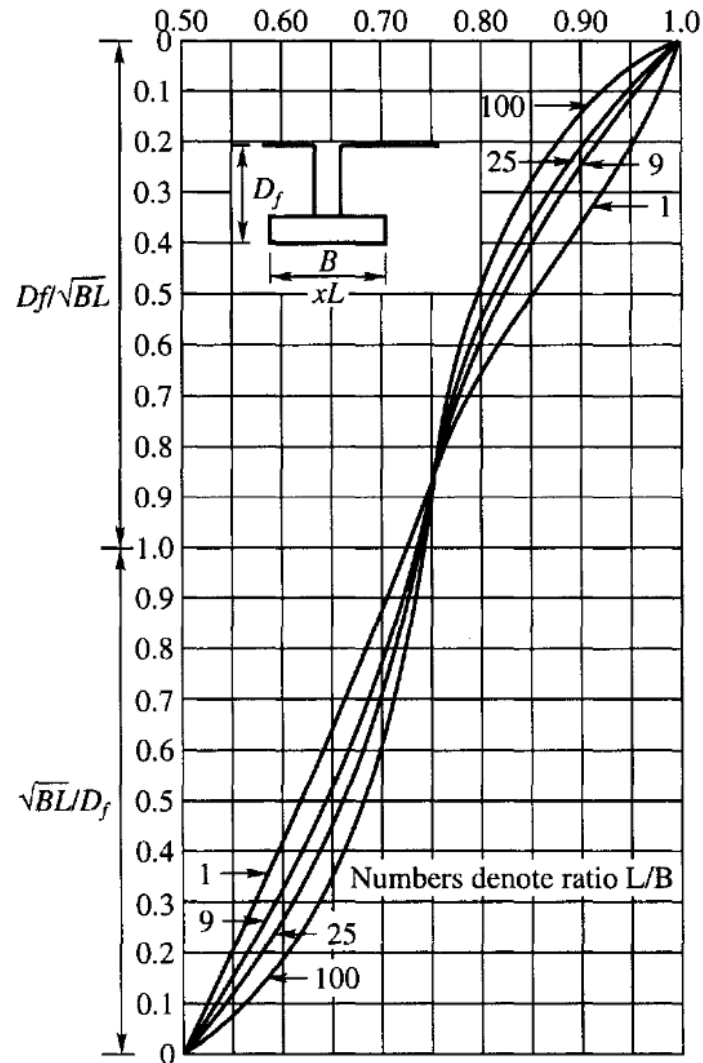
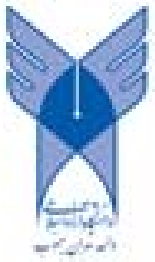


Figure 13.8 Correction curves for elastic settlement of flexible rectangular foundations at depth (Fox, 1948)



13.11 JANBU, BJERRUM AND KJAERNSLI'S METHOD OF DETERMINING ELASTIC SETTLEMENT UNDER UNDRAINED CONDITIONS

$$S_e = \frac{\mu_0 \mu_1 q_n B}{E_s} \quad (13.22)$$

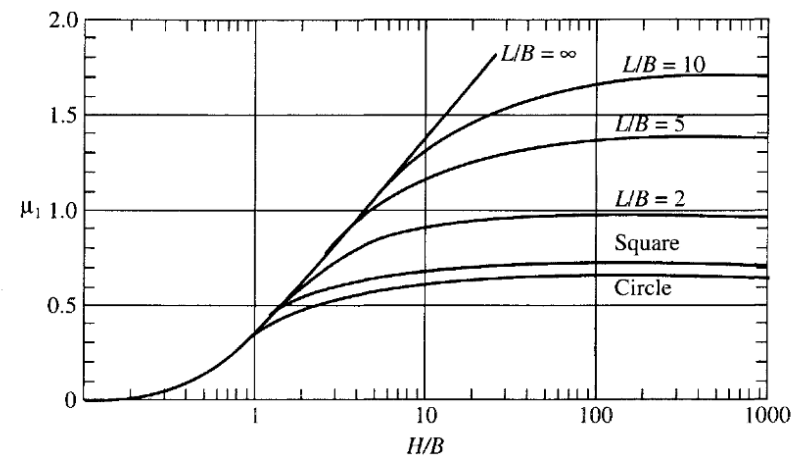
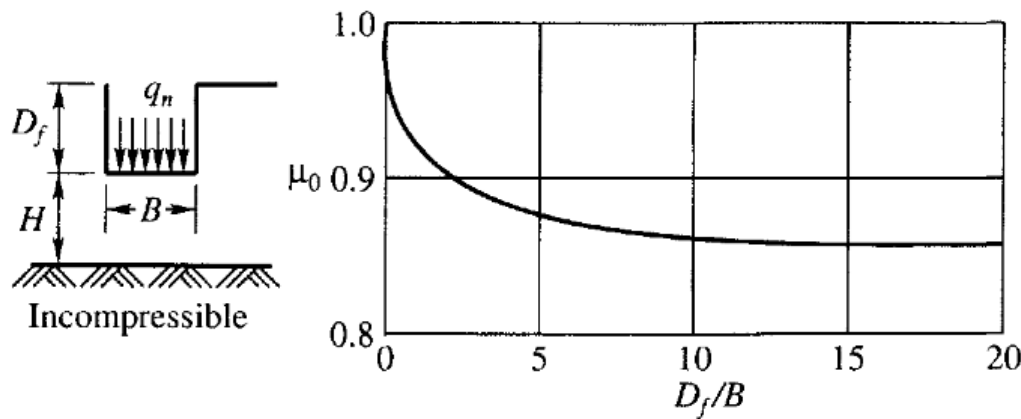
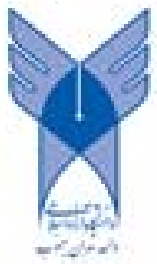


Figure 13.9 Factors for calculating the average immediate settlement of a loaded area (after Christian and Carrier, 1978)



13.12 SCHMERTMANN'S METHOD OF CALCULATING SETTLEMENT IN GRANULAR SOILS BY USING CPT VALUES

$$S = C_1 C_2 q_n \sum_0^{2B} \frac{I_z}{E_s} \Delta z \quad (13.23)$$

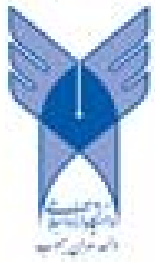
- where, S = total settlement,
 q_n = net foundation base pressure = $(q - q'_0)$,
 q = total foundation pressure,
 q'_0 = effective overburden pressure at foundation level,
 Δz = thickness of elemental layer,
 I_z = vertical strain influence factor,
 C_1 = depth correction factor,
 C_2 = creep factor.

The equations for C_1 and C_2 are

$$C_1 = 1 - 0.5 \frac{q'_0}{q_n} \quad (13.24)$$

$$C_2 = 1 + 0.2 \log_{10} \frac{t}{0.1} \quad (13.25)$$

where t is time in years for which period settlement is required.



Example 13.8

Estimate the immediate settlement of a concrete footing 1.5×1.5 m in size founded at a depth of 1 m in silty soil whose modulus of elasticity is 90 kg/cm^2 . The footing is expected to transmit a unit pressure of 200 kN/m^2 .

Solution

Use Eq. (13.20a)

Immediate settlement,

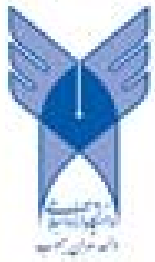
$$S_e = qB \frac{(1 - \mu^2)}{E_s} I_f$$

Assume $\mu = 0.35$, $I_f = 0.82$ for a rigid footing.

Given: $q = 200 \text{ kN/m}^2$, $B = 1.5 \text{ m}$, $E_s = 90 \text{ kg/cm}^2 \approx 9000 \text{ kN/m}^2$.

By substituting the known values, we have

$$S_e = 200 \times 1.5 \times \frac{1 - 0.35^2}{9000} \times 0.82 = 0.024 \text{ m} = 2.4 \text{ cm}$$



Example 13.9

A square footing of size 8×8 m is founded at a depth of 2 m below the ground surface in loose to medium dense sand with $q_n = 120$ kN/m². Standard penetration tests conducted at the site gave the following corrected N_{60} values.

Depth below G.L. (m)	N_{cor}	Depth below G.L.	N_{cor}
2	8	10	11
4	8	12	16
6	12	14	18
8	12	16	17
		18	20

The water table is at the base of the foundation. Above the water table $\gamma = 16.5$ kN/m³, and submerged $\gamma_b = 8.5$ kN/m³.

Compute the elastic settlement by Eq. (13.20a). Use the equation $E_s = 250 (N_{cor} + 15)$ for computing the modulus of elasticity of the sand. Assume $\mu = 0.3$ and the depth of the compressible layer = $2B = 16$ m ($= H$).

Solution

For computing the elastic settlement, it is essential to determine the weighted average value of N_{cor} . The depth of the compressible layer below the base of the foundation is taken as equal to 16 m ($= H$). This depth may be divided into three layers in such a way that N_{cor} is approximately constant in each layer as given below.



Layer No.	Depth (m)		Thickness (m)	N_{cor}
	From	To		
1	2	5	3	9
2	5	11	6	12
3	11	18	7	17

The weighted average

$$N_{cor}(av) = \frac{9 \times 3 + 12 \times 6 + 17 \times 7}{16} = 13.6 \text{ or say } 14$$

From equation $E_s = 250 (N_{cor} + 15)$ we have

$$E_s = 250(14 + 15) = 7250 \text{ kN/m}^2$$

The total settlement of the center of the footing of size 8×8 m is equal to four times the settlement of a corner of a footing of size 4×4 m.

In the Eq. (13.20a), $B = 4$ m, $q_n = 120 \text{ kN/m}^2$, $\mu = 0.3$.

Now from Fig. 13.7, for $H/B = 16/4 = 4$, $L/B = 1$

$$F_1 = I_f = 0.4 \text{ for } \mu = 0.5$$

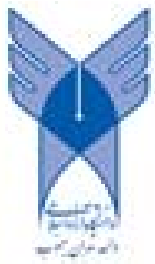
$$F_2 = 0.03 \text{ for } \mu = 0.5$$

Now from Eq. (13.20 b) I_f for $\mu = 0.3$ is

$$I_f = F_1 + \frac{(1 - \mu - 2\mu^2)F_2}{1 - \mu^2} = 0.40 + \frac{(1 - 0.3 - 2 \times 0.3^2) \times 0.03}{1 - 0.3^2} = 0.42$$

From Eq. (13.20a) we have settlement of a corner of a footing of size 4×4 m as

$$S_e = q_n B \frac{(1 - \mu^2)}{E_s} I_f = \frac{120 \times 4(1 - 0.3^2)}{7250} \times 0.42 \times 100 = 2.53 \text{ cm}$$



With the correction factor, the final elastic settlement from Eq. (13.21) is

$$S_{ef} = C_r d_f S_e$$

where C_r = rigidity factor = 1 for flexible footing d_f = depth factor

From Fig. 13.8 for

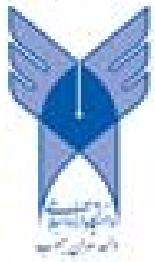
$$\frac{D_f}{\sqrt{BL}} = \frac{2}{\sqrt{4 \times 4}} = 0.5, \quad \frac{L}{B} = \frac{4}{4} = 1 \quad \text{we have } d_f = 0.85$$

$$\text{Now } S_{ef} = 1 \times 0.85 \times 2.53 = 2.15 \text{ cm}$$

The total elastic settlement of the center of the footing is

$$S_e = 4 \times 2.15 = 8.6 \text{ cm} = 86 \text{ mm}$$

Per Table 13.1a, the maximum permissible settlement for a raft foundation in sand is 62.5 mm. Since the calculated value is higher, the contact pressure q_n has to be reduced.



Example 13.10

It is proposed to construct an overhead tank at a site on a raft foundation of size 8×12 m with the footing at a depth of 2 m below ground level. The soil investigation conducted at the site indicates that the soil to a depth of 20 m is normally consolidated insensitive inorganic clay with the water table 2 m below ground level. Static cone penetration tests were conducted at the site using a mechanical cone penetrometer. The average value of cone penetration resistance \bar{q}_c was found to be 1540 kN/m^2 and the average saturated unit weight of the soil = 18 kN/m^3 . Determine the immediate settlement of the foundation using Eq. (13.22). The contact pressure $q_n = 100 \text{ kN/m}^2$ ($= 0.1 \text{ MPa}$). Assume that the stratum below 20 m is incompressible.

Solution

Computation of the modulus of elasticity

Use Eq. (13.19) with $A = 500$

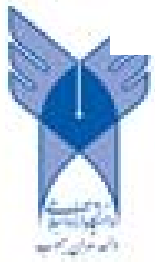
$$E_s = 500 c_u$$

where c_u = the undrained shear strength of the soil

From Eq. (9.14)

$$c_u = \frac{\bar{q}_c - p_o}{N_k}$$

where \bar{q}_c = average static cone penetration resistance = 1540 kN/m^2
 p_o = average total overburden pressure = $10 \times 18 = 180 \text{ kN/m}^2$
 N_k = 20 (assumed)



$$\text{Therefore } c_u = \frac{1540 - 180}{20} = 68 \text{ kN/m}^2$$

$$E_s = 500 \times 68 = 34,000 \text{ kN/m}^2 = 34 \text{ MPa}$$

Eq. (13.22) for S_e is

$$S_e = \frac{\mu_0 \mu_1 q_n B}{E_s}$$

From Fig. 13.9 for $D/B = 2/8 = 0.25$, $\mu_0 = 0.95$, for $H/B = 16/8 = 2$ and $L/B = 12/8 = 1.5$, $\mu_1 = 0.6$.
Substituting

$$S_e (\text{average}) = \frac{0.95 \times 0.6 \times 0.1 \times 8}{34} = 0.0134 \text{ m} = 13.4 \text{ mm}$$

From Fig. 13.8 for $D_f / \sqrt{BL} = 2 / \sqrt{8 \times 12} = 0.2$, $L/B = 1.5$ the depth factor $d_f = 0.94$

The corrected settlement S_{ef} is

$$S_{ef} = 0.94 \times 13.4 = 12.6 \text{ mm}$$



13.13 ESTIMATION OF CONSOLIDATION SETTLEMENT BY USING OEDOMETER TEST DATA

Normally consolidated clays

$$S_c = H \frac{C_c}{1+e_0} \log \frac{p_0 + \Delta p}{p_0} \quad (13.33)$$

Overconsolidated clays

for $p_0 + \Delta p < p_c$

$$S_c = H \frac{C_s}{1+e_0} \log \frac{p_0 + \Delta p}{p_0} \quad (13.34)$$

for $p_0 < p_c < p_0 + \Delta p$

$$S_c = \frac{H}{1+e_0} C_s \log \frac{p_c}{p_0} + C_c \log \frac{p_0 + \Delta p}{p_c} \quad (13.35)$$

where C_s = swell index, and C_c = compression index



Example 13.12

For the problem given in Ex. 13.10 compute the consolidation settlement by the Skempton-Bjerrum method. The compressible layer of depth 16 m below the base of the foundation is divided into four layers and the soil properties of each layer are given in Fig. Ex. 13.12. The net contact pressure $q_n = 100 \text{ kN/m}^2$.

Solution

From Eq. (13.33), the oedometer settlement for the entire clay layer system may be expressed as

$$S_{oe} = H_i \frac{C_c}{1+e_o} \log \frac{p_o + \Delta p}{p_o}$$

From Eq. (13.41), the consolidation settlement as per Skempton-Bjerrum may be expressed as

$$S_c = \beta S_{oe}$$

where β = settlement coefficient which can be obtained from Fig. 13.12 for various values of A and H/B .

p_o = effective overburden pressure at the middle of each layer (Fig. Ex. 13.12)

C_c = compression index of each layer

H_i = thickness of i th layer

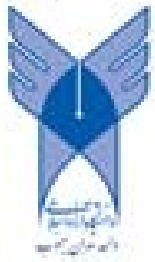
e_o = initial void ratio of each layer

Δp = the excess pressure at the middle of each layer obtained from elastic theory (Chapter 6)

The average pore pressure coefficient is

$$A = \frac{0.9 + 0.75 + 0.70 + 0.45}{4} = 0.7$$

The details of the calculations are tabulated below.



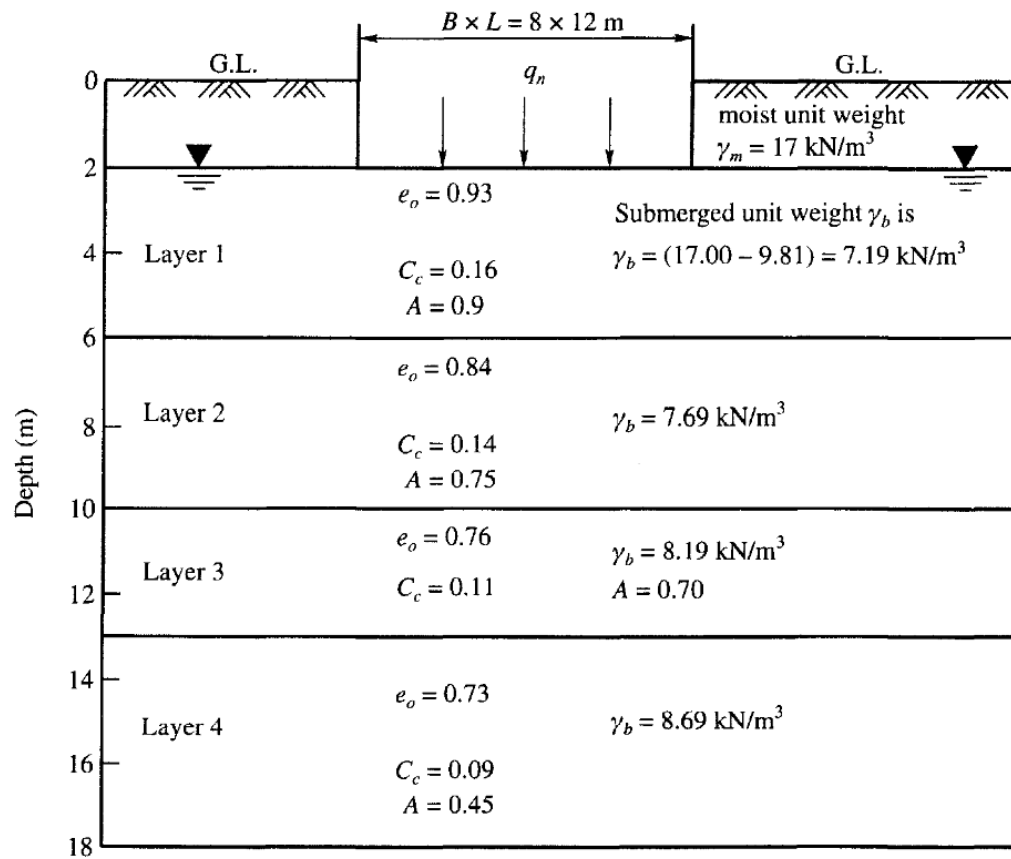


Figure Ex. 13.12

Layer No.	H_i (cm)	p_o (kN/m ²)	Δp (kN/m ²)	C_c	e_o	$\log \frac{p_o + \Delta p}{p_o}$	S_{oed} (cm)
1	400	48.4	75	0.16	0.93	0.407	13.50
2	400	78.1	43	0.14	0.84	0.191	5.81
3	300	105.8	22	0.11	0.76	0.082	1.54
4	500	139.8	14	0.09	0.73	0.041	1.07
Total							21.92

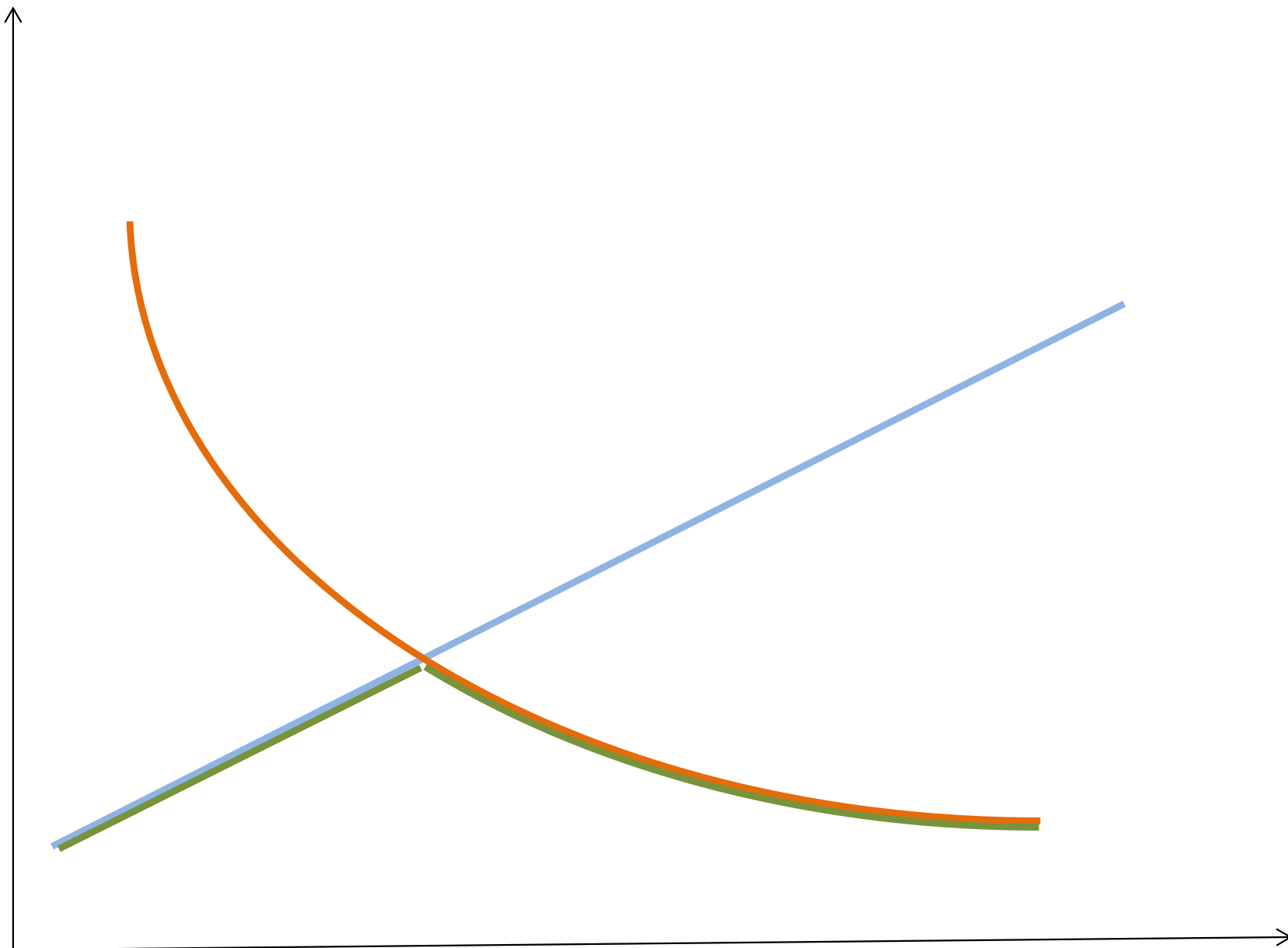
For $H/B = 16/8 = 2$, $A = 0.7$, from Fig. 13.12 we have $\beta = 0.8$.

The consolidation settlement S_c is

$$S_c = 0.8 \times 21.92 = 17.536 \text{ cm} = 175.36 \text{ mm}$$



Q_{all}



B



تمرین های حل شده



EXAMPLE 12.6 Factor of Safety of a Footing Subjected to a Vertical Load and a Moment

The footing in Example 12.1 is subjected to a vertical load of 500 kN and a moment about the Y axis of 125 kN.m. Calculate the factor of safety.

Strategy Since we are only given the moment about the Y axis, we only need to find the eccentricity, e_B . The bearing capacity factors are the same as those in Example 12.1.

Solution 12.6

Step 1: Draw a sketch of the problem and calculate e_B .

See Figure E12.6 for a sketch.

$$M_y = 125 \text{ kN.m}, \quad P = 500 \text{ kN}; \quad e_B = \frac{M_y}{P} = \frac{125}{500} = 0.25 \text{ m}$$

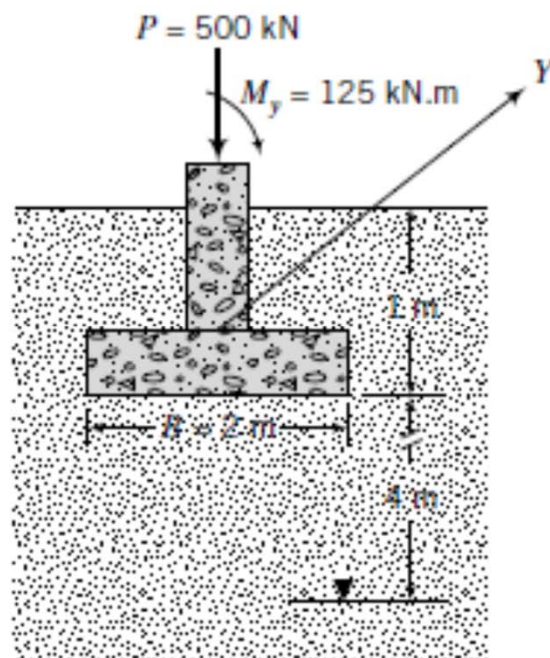


FIGURE E12.6

Step 2: Check if tension develops.

$$\frac{B}{6} = \frac{2}{6} = 0.33 \text{ m} > e_B = 0.25 \text{ m}$$

Therefore, tension will not occur.

Step 3: Calculate the maximum vertical stress.

$$\sigma_{max} = \frac{P}{BL} \left(1 + \frac{6e_B}{B} \right) = \frac{1000}{2 \times 2} \left(1 + \frac{6 \times 0.25}{2} \right) = 438 \text{ kPa}$$

Step 4: Calculate reduced footing size.

$$B' = 2 - 2(0.25) = 1.5 \text{ m}$$

Step 5: Calculate the shape and depth factors.

$$s_q = 1 + \frac{B'}{L} \tan \phi'_p = 1 + \frac{1.5}{2} \tan 35^\circ = 1.53$$

$$s_\gamma = 1 - 0.4 \frac{B'}{L} = 1 - 0.4 \frac{1.5}{2} = 0.7$$

$$d_q = 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \left(\frac{D_f}{B'} \right) = 1 + 2 \tan 35^\circ (1 - \sin 35^\circ)^2 \left[\tan^{-1} \left(\frac{1}{1.5} \times \frac{\pi}{180} \right) \right] = 1.15$$

$$d_\gamma = 1$$

Step 6: Substitute the appropriate values into the bearing capacity equation.

$$\begin{aligned} q_u &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 32.3 \times 1.53 \times 1.15) + (0.5 \times 18 \times 1.5 \times 37.1 \times 0.7 \times 1.0) \\ &= 1373 \text{ kPa} \end{aligned}$$

Step 7: Calculate the factor of safety.

$$FS = \frac{q_u}{(\sigma_a)_{max} - \gamma D_f} = \frac{1373}{438 - 1 \times 18} = 3.3$$

EXAMPLE 12.1 Allowable Bearing Capacity of a Sand

A footing 2 m square, subjected to a centric vertical load, is located at a depth of 1.0 m below the ground surface in a deep deposit of compacted sand ($\phi'_p = 35^\circ$, $\phi'_{cs} = 30^\circ$, and $\gamma_{sat} = 18 \text{ kN/m}^3$). The groundwater level is 5 m below the ground surface, but you should assume that the soil above the groundwater is saturated. The friction angles were obtained from plain strain tests. Determine the allowable bearing capacity for a factor of safety of 3.

Strategy It is a good policy to sketch a diagram illustrating the conditions given (see Figure E12.1). The groundwater level is located at $(5 \text{ m} - 1 \text{ m}) = 4 \text{ m}$ from the footing base. That is, the groundwater level is more than $B = 2 \text{ m}$ below the base. We can neglect the effects of groundwater.

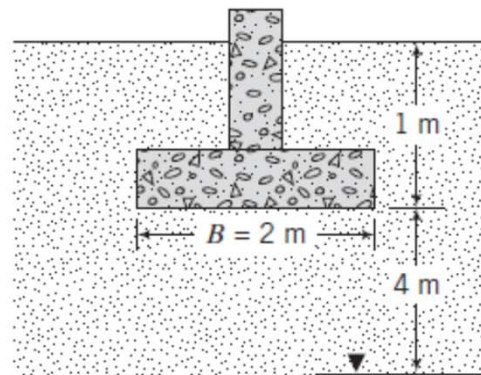


FIGURE E12.1

Solution 12.1

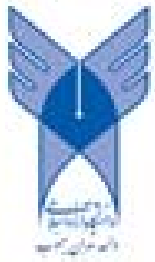
Step 1: Calculate the bearing capacity factors and geometric factors. Assume rough footing. Use $\phi' = \phi'_p = 35^\circ$.

The eccentricity is zero, so $B' = B$ and $L' = L$.

$$N_q = e^{\pi \tan \phi'_p} \tan^2 \left(45^\circ + \frac{\phi'_p}{2} \right) = e^{\pi \tan 35^\circ} \tan^2 \left(45^\circ + \frac{35^\circ}{2} \right) = 33.3$$

$$N_q - 1 = 32.3$$

$$N_\gamma = 0.1054 \exp(9.6 \phi'_p) = 0.1054 \exp \left(9.6 \times \frac{35 \times \pi}{180} \right) = 37.1$$



$$s_q = 1 + \frac{B'}{L'} \tan \phi'_p = 1 + \frac{2}{2} \tan 35^\circ = 1.70$$

$$s_\gamma = 1 - 0.4 \frac{B'}{L'} = 1 - 0.4 \frac{2}{2} = 0.6$$

$$d_\gamma = 1.0$$

$$d_q = 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \left(\frac{D_f}{B'} \right)$$

$$= 1 + (2 \tan 35^\circ) (1 - \sin 35^\circ)^2 \left[\tan^{-1} \left(\frac{1}{2} \right) \times \frac{\pi}{180} \right] = 1.13$$

Step 2: Calculate the ultimate net bearing capacity.

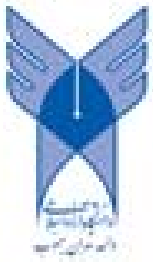
$$q_u = \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B N_\gamma s_\gamma d_\gamma$$

$$q_u = (18 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 18 \times 2 \times 37.1 \times 0.6 \times 1.0)$$

$$= 1515 \text{ kPa}$$

$$q_a = \frac{q_u}{\text{FS}} + \gamma D_f$$

$$= \frac{1515}{3} + 18 \times 1 = 523 \text{ kPa}$$



EXAMPLE 12.2 *The Effects of Groundwater on Bearing Capacity*

Compare the ultimate net bearing capacity for Example 12.1 using $\phi'_p = 35^\circ$ when the groundwater is located (a) at 5 m below the ground surface, (b) at the ground surface, (c) at the bottom of the base of the footing, and (d) at 1 m below the base.

Strategy The trick here is to use the appropriate value of the unit weight in the bearing capacity equation.

Solution 12.2

Step 1: Calculate bearing capacity numbers and shape and depth factors. These values are the same as in Example 12.1.

Step 2: Substitute values from Step 1 into Equation (12.14).

(a) *Groundwater level at 5 m below the surface.* The groundwater level is 4 m below the base, which is greater than the width of the footing. Therefore, groundwater has no effect.

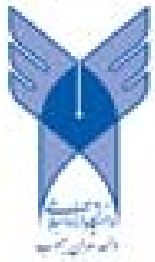
$$\text{From Example 12.1: } q_u = 1515 \text{ kPa}$$

(b) *Groundwater level at the ground surface.* In this case, the groundwater level will affect the bearing capacity. You should use

$$\begin{aligned}\gamma' &= \gamma_{sat} - \gamma_w = 18 - 9.8 = 8.2 \text{ kN/m}^3 \\ q_u &= \gamma' D_f (N_q - 1) s_q d_q + 0.5 \gamma' B' N_{\gamma} s_{\gamma} d_{\gamma} \\ &= (8.2 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 8.2 \times 2 \times 37.1 \times 0.6 \times 1.0) \\ &= 691 \text{ kPa}\end{aligned}$$

Alternatively, since the change in the unit weight is the same for both terms of the bearing capacity equation, we can simply find q_u by taking the ratio γ'/γ , that is,

$$q_u = 1515 \times \frac{8.2}{18} = 690 \text{ kPa}$$



(c) *Groundwater level at the bottom of the base.* In this case, the groundwater level will affect the last term in the bearing capacity.

Thus,

$$\begin{aligned} q_u &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma' B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 8.2 \times 2 \times 37.1 \times 0.6 \times 1.0) \\ &= 1299 \text{ kPa} \end{aligned}$$

(d) *Groundwater level at 1 m below the bottom of the base.* In this case, the groundwater level is within a depth B below the base and will affect the last term in the bearing capacity, where you should use

$$\gamma' B' = \gamma_{sat} z + \gamma' (B' - z) = 18 \times 1 + 8.2 \times (2 - 1) = 26.2 \text{ kN/m}^2$$

Thus,

$$\begin{aligned} q_u &= \gamma D_f (N_q - 1) s_q d_q + 0.5 (\gamma' B') N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 32.3 \times 1.7 \times 1.13) + (0.5 \times 26.2 \times 37.1 \times 0.6 \times 1.0) \\ &= 1408 \text{ kPa} \end{aligned}$$

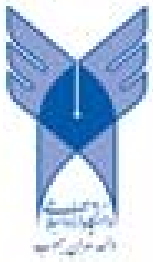
Step 3: Compare results.

We will compare the results by dividing (normalizing) each ultimate net bearing capacity by the ultimate net bearing capacity of case (a).

Groundwater level at	$\frac{q_u}{(q_u)_{(a)}} \times 100$
(b) Ground surface	$\frac{691}{1515} \times 100 \approx 46\%$
(c) Base	$\frac{1299}{1515} \times 100 = 86\%$
(d) 1 m below base	$\frac{1408}{1515} \times 100 = 93\%$

Note: $(q_u)_{(a)}$ is the net ultimate bearing capacity for case (a).

The groundwater level rising to the surface will reduce the bearing capacity by more than one-half.



EXAMPLE 12.3 Allowable Short-term Bearing Capacity of a Clay Soil

A footing $1.8 \text{ m} \times 2.5 \text{ m}$ is located at a depth of 1.5 m below the ground surface in a deep deposit of a saturated overconsolidated clay. The groundwater level is 2 m below the ground surface. The undrained shear strength from a direct simple shear test is 120 kPa and $\gamma_{sat} = 20 \text{ kN/m}^3$. Determine the allowable bearing capacity, assuming a factor of safety of 3, for short-term condition. Neglect the effects of embedment.

Strategy Use the equation for the short-term bearing capacity. You do not need to consider the effect of groundwater when you are evaluating short-term condition.

Solution 12.3

Step 1: Calculate geometric factors.

No eccentricity: $B' = B, L' = L$

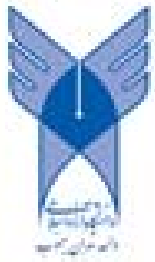
$$s_c = 1 + 0.2 \frac{B'}{L'} = 1 + 0.2 \frac{1.8}{2.5} = 1.14, \quad d_c = 1$$

Step 2: Calculate q_u .

$$q_u = 5.14 s_u s_c d_c = 5.14 \times 120 \times 1.14 \times 1 = 703 \text{ kPa}$$

Step 3: Calculate q_a .

$$q_a = \frac{q_u}{FS} + \gamma D_f = \frac{703}{3} + 1.5 \times 20 = 264 \text{ kPa}$$



EXAMPLE 12.4 Sizing a Rectangular Footing Using ASD and LRFD

Determine the size of a rectangular footing to support vertical centric dead and live loads of 800 kN and 1000 kN, respectively, on a dense, coarse-grained soil. The friction angle obtained from a triaxial test is $\phi'_p = 28.4^\circ$ and $\gamma_{sat} = 18 \text{ kN/m}^3$. The footing is to be located at 1 m below the ground surface. Groundwater level is 6 m below the ground surface. Assume $FS = 3$, $\eta_f = 1$, and $\varphi = 0.8$.

Strategy Neither the footing width nor the length is given. Both of these are required to find q_u . You can fix a length-to-width ratio and then assume a width (B). Solve for q_u , and if it is not satisfactory [$q_u \geq (\sigma)_{max}$], then reiterate using a different B value. You have to convert the triaxial friction angle to an equivalent plane strain value using Equation (12.24).

Solution 12.4

Step 1: Calculate bearing capacity numbers, shape, and depth factors.

Assume $B' = B = 1.5 \text{ m}$ and $\frac{L'}{B'} = 1.5$; that is, $L' = 1.5 \times 1.5 = 2.25 \text{ m}$ and $\frac{B'}{L'} = \frac{1.5}{2.25} = 0.67 \text{ m}$. Footing area $A = B'L' = 1.5 \times 2.25 = 3.375 \text{ m}^2$.

$$\begin{aligned}(\phi'_p)_{ps} &= \frac{9}{8}(\phi'_p)_{tr} = \frac{9}{8} \times 28.4^\circ = 32^\circ \\ N_q &= e^{\pi \tan 32^\circ} \tan^2(45^\circ + 32^\circ/2) = 23.2 \\ N_q - 1 &= 23.2 - 1 = 22.2\end{aligned}$$

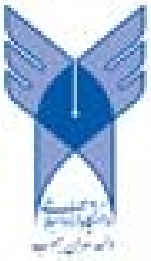
Assume rough footing.

$$\begin{aligned}N_\gamma &= 0.1054 \exp(9.6\phi'_p) = 0.1054 \exp\left(9.6 \times \frac{32 \times \pi}{180}\right) \\ &= 22.5 \\ s_q &= 1 + \frac{B'}{L'} \tan \phi'_p = 1 + 0.67 \tan 32^\circ = 1.42 \\ s_\gamma &= 1 - 0.4 \frac{B'}{L'} = 0.73 \\ d_q &= 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \frac{D_f}{B'} \\ &= 1 + (2 \tan 32^\circ)(1 - \sin 32^\circ)^2 \left[\tan^{-1} \left(\frac{1}{1.5} \right) \times \frac{\pi}{180} \right] \\ &= 1.18 \\ d_\gamma &= 1.0\end{aligned}$$

Step 2: Calculate the ultimate and allowable bearing capacity.

Substitute the values in Step 1 into the bearing capacity equation, Equation (12.16). The groundwater level is located more than B below the base. Therefore, groundwater will not affect the bearing capacity.

$$\begin{aligned}q_u &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 22.2 \times 1.42 \times 1.18) + (0.5 \times 18 \times 1.5 \times 22.5 \times 0.73 \times 1.0) = 891 \text{ kPa}\end{aligned}$$



$$q_{ult} = q_u + \gamma D_f = 891 + 18 \times 1 = 909 \text{ kPa}$$

$$q_a = \frac{q_u}{FS} + \gamma D_f = \frac{891}{3} + 18 \times 1 = 315 \text{ kPa}$$

$$R = q_{ult} \times A = 909 \times 3.375 = 3068 \text{ kN}$$

$$P_a = q_a \times A = 315 \times 3.375 = 1063 \text{ kN}$$

Step 3: Calculate the imposed stress based on ASD and LRFD.

$$\text{ASD: } P = DL + LL = 800 + 1000 = 1800 \text{ kN}$$

$$\text{LRFD: } P_{uf} = 1.25 DL + 1.75 LL = 1.25 \times 800 + 1.75 \times 1000 = 2750 \text{ kN}$$

The term P_{uf} is the factored load.

Step 4: Check suitability of assumed foundation size.

$$\text{LRFD: } \phi R = 0.8 \times 3068 = 2454 \text{ kN} < P_{uf} (= 2750 \text{ kN})$$

Unacceptable; try another footing size.

$$\text{ASD: } P_a (= 1063 \text{ kN}) < P (= 1800 \text{ kN})$$

Unacceptable; try another footing size.

Step 5: Try another width and recalculate.

We need to try a larger B , keeping $\frac{L'}{B'} = 1.5$. Try $B = B' = 2$ m. The depth factor for this case changes to 1.14. s_γ and s_q have the same values.

$$\begin{aligned} q_{ult} &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 22.2 \times 1.42 \times 1.14) + (0.5 \times 18 \times 2 \times 22.5 \times 0.73 \times 1.0) + 18 \times 1 \\ &= 942 + 18 = 960 \text{ kPa} \end{aligned}$$

$$q_a = \frac{q_u}{FS} + \gamma D_f = \frac{942}{3} + 18 \times 1 = 332 \text{ kPa}$$

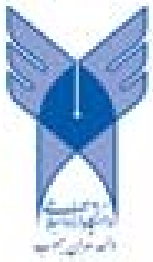
$$R = q_{ult} \times A = 960 \times (2 \times 3) = 5760 \text{ kN}$$

$$P_a = q_a \times A = 332 \times (2 \times 3) = 1992 \text{ kN}$$

Step 6: Check suitability of assumed foundation size.

$$\text{LRFD: } \phi R = 0.8 \times 5760 = 4608 \text{ kN} > P_{uf} (= 2750 \text{ kN}); \text{ acceptable.}$$

$$\text{ASD: } P_a (= 1992 \text{ kN}) > P (= 1800 \text{ kN}); \text{ acceptable.}$$



EXAMPLE 12.5 Allowable Bearing Capacity Due to an Inclined Load

Using the footing geometry of Example 12.1, determine q_a for a load inclined at 20° to the vertical along the footing width (see Figure E12.5).

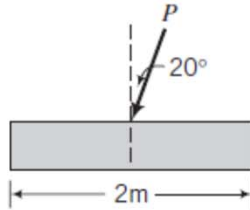


FIGURE E12.5

Strategy You need to use Equation (12.18) for inclined loads. You only need to calculate the inclination factors, since shape and depth factors are not included for load inclination.

Solution 12.5

Step 1: Calculate the inclination factors and depth factors.

$$B' = B; \quad \omega = 20^\circ$$

$$n = n_B = \left(2 + \frac{B'}{L'}\right) \left(1 + \frac{B'}{L'}\right) = (2 + 1)/(1 + 1) = 1.5$$

$$i_q = \left(1 - \frac{H}{V_n}\right)^n = (1 - \tan \omega)^n = (1 - \tan 20^\circ)^{1.5} = 0.51$$

$$i_y = \left(1 - \frac{H}{V_n}\right)^{n-1} = (1 - \tan \omega)^{n+1} = (1 - \tan 20^\circ)^{1.5+1} = 0.32$$

Step 2: Calculate the ultimate net bearing capacity and allowable bearing capacity.

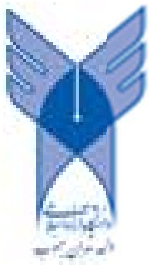
Use Equation (12.18).

$$q_u = \gamma D_f (N_q - 1) i_q + 0.5 \gamma B N_\gamma i_\gamma = (18 \times 1 \times 32.3 \times 0.51) + (0.5 \times 18 \times 2 \times 37.1 \times 0.32) = 510 \text{ kPa}$$

$$q_a = \frac{q_u}{\text{FS}} + \gamma D_f = \frac{510}{3} + 18 \times 1 = 188 \text{ kPa}$$

The allowable bearing capacity for a vertical centric load is 523 kPa from Example 12.1.

Reduction in allowable bearing capacity is $\frac{523 - 188}{523} = 0.64 = 64\%$.



EXAMPLE 12.6 Factor of Safety of a Footing Subjected to a Vertical Load and a Moment

The footing in Example 12.1 is subjected to a vertical load of 500 kN and a moment about the Y axis of 125 kN.m. Calculate the factor of safety.

Strategy Since we are only given the moment about the Y axis, we only need to find the eccentricity, e_B . The bearing capacity factors are the same as those in Example 12.1.

Solution 12.6

Step 1: Draw a sketch of the problem and calculate e_B .

See Figure E12.6 for a sketch.

$$M_y = 125 \text{ kN.m}, \quad P = 500 \text{ kN}; \quad e_B = \frac{M_y}{P} = \frac{125}{500} = 0.25 \text{ m}$$

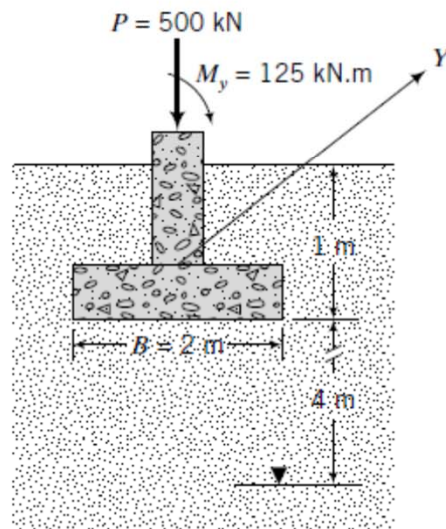
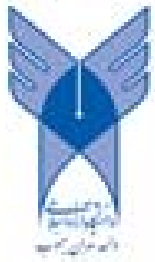


FIGURE E12.6



Step 2: Check if tension develops.

$$\frac{B}{6} = \frac{2}{6} = 0.33 \text{ m} > e_B = 0.25 \text{ m}$$

Therefore, tension will not occur.

Step 3: Calculate the maximum vertical stress.

$$\sigma_{max} = \frac{P}{BL} \left(1 + \frac{6e_B}{B} \right) = \frac{1000}{2 \times 2} \left(1 + \frac{6 \times 0.25}{2} \right) = 438 \text{ kPa}$$

Step 4: Calculate reduced footing size.

$$B' = 2 - 2(0.25) = 1.5 \text{ m}$$

Step 5: Calculate the shape and depth factors.

$$s_q = 1 + \frac{B'}{L} \tan \phi'_p = 1 + \frac{1.5}{2} \tan 35^\circ = 1.53$$

$$s_\gamma = 1 - 0.4 \frac{B'}{L} = 1 - 0.4 \frac{1.5}{2} = 0.7$$

$$d_q = 1 + 2 \tan \phi'_p (1 - \sin \phi'_p)^2 \tan^{-1} \left(\frac{D_f}{B'} \right) = 1 + 2 \tan 35^\circ (1 - \sin 35^\circ)^2 \left[\tan^{-1} \left(\frac{1}{1.5} \times \frac{\pi}{180} \right) \right] = 1.15$$

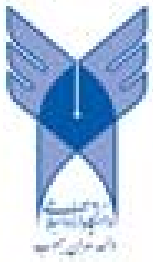
$$d_\gamma = 1$$

Step 6: Substitute the appropriate values into the bearing capacity equation.

$$\begin{aligned} q_u &= \gamma D_f (N_q - 1) s_q d_q + 0.5 \gamma B' N_\gamma s_\gamma d_\gamma \\ &= (18 \times 1 \times 32.3 \times 1.53 \times 1.15) + (0.5 \times 18 \times 1.5 \times 37.1 \times 0.7 \times 1.0) \\ &= 1373 \text{ kPa} \end{aligned}$$

Step 7: Calculate the factor of safety.

$$FS = \frac{q_u}{(\sigma_a)_{max} - \gamma D_f} = \frac{1373}{438 - 1 \times 18} = 3.3$$





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