UNIVERSITY OF JOHANNESBURG

PROGRAM	:	BACHELOR OF ENGINEERING SCIENCE *CIVIL ENGINEERING*
SUBJECT	:	**GEOTECHNICAL ENGINEERING 4A**
CODE	:	**GTG4A11**
DATE	:	MAIN EXAMINATION 28 MAY 2019
DURATION	:	12:30 - 15:30
WEIGHT	:	50:50
TOTAL MARKS	:	100

EXAMINER	:	PROF FN OKONTA / MR FA TCHANI
MODERATOR	:	MED KWESIGA
NUMBER OF PAGES	:	17 PAGES

INSTRUCTIONS	:	QUESTION PAPERS MUST BE HANDED IN. CLOSED BOOK EXAMINATION

INSTRUCTIONS TO CANDIDATES:

PLEASE ANSWER ALL THE QUESTIONS.
PLEASE NUMBER ALL QUESTIONS EXACTLY AS QUESTION PAPER

SHOW ALL CALCULATIONS

QUESTION 1 [20 MARKS]

1.1 A circular footing was chosen as a spread footing foundation to transmit a structural load of 360 KN. The circular footing of a diameter of 1.3 m is to be embedded in a silty sand at a depth of 0.8 m from the ground. Laboratory tests performed on a soil sample revealed the following properties of the soil

In situ unit weight of the soil $\gamma_{in\text{-}situ}$= 19.5 KN/m^3, Saturated unit weight of the soil $\gamma_{saturated\ unit\ weight}$ = 21.6 KN/m^3. Angle of friction Ø' = 30°; c' = 0.

a. Calculate the factor of safety against a bearing capacity failure if the ground water table is found at 8 m from the ground surface. [7]

b. If a sudden flooding raises the water table to 1 m below the ground surface, calculate the new factor of safety against a bearing capacity failure. [7]

1.2 Discuss the effect of the rise of the water table on the stability of a shallow foundation and buried steel tank, when empty and when filled with water [6]

QUESTION 2 [15 MARKS]

2.1 Discuss the conditions that will determine your choice between selecting a pile foundation or a raft foundation for a multi-story building. [5]

An entire building has to transmit 3000 KN to the ground through square spread footings of 1.5 m width embedded to 0.8 m. If the soil profile has the following characteristics:

C_c/ (1+e0) =0.13, C_r/(1+e0) = 0.025, γ= 19.6 KN/m^3, σ'_m = 200 KPa, no cohesion, ϕ'=30.

2.2 What is the minimum ***even*** number of columns the entire building must have for the strength and the serviceability requirements of the foundation to be met taking 3 as a factor of safety? [10]

QUESTION 3 [20 MARKS]

3.1 Discuss the major differences between the Meyerhof and Vesic methods of quantifying pile capacity. [6]

3.2 A 25 m long concrete pile with a cross sectional area of 0.25m^2 fully embedded in a medium dense sandy soil formation. The sandy profile has a unit weight, γ = 18 kN/m^3 and as soil friction angle ϕ'= 32^o. Estimate the ultimate point bearing capacity of the pile Q_p with each of the following methods:

a. Meyerhof's method
b. Vesic's method

Based on the results obtained in a and b above, recommend a Q_p value that will be used for design. [14]

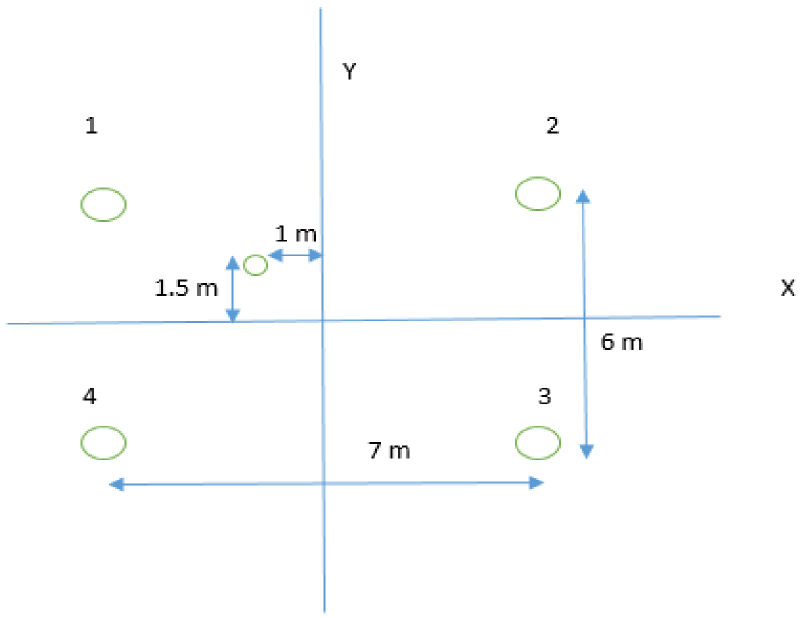
QUESTION 4 [25 MARKS]

4.1 Explain, the efficiency of a pile group in sand and in clay formation. [5]

4.2A group of 4 concrete piles of 0.6 m of diameter is arranged as depicted in the picture below. The piles are 20 m long and are driven in soft clay with an unconfined compressive strength of 48 KN/m^3. The bearing resistance may be neglected for the piles and the factor of safety taken as 2.

a. What is the maximum safe load that can be placed on the PILE CAP? [8]

b. What is the load each pile is experiencing? (Assume an efficiency of 1 for the pile group). [12]



QUESTION 5[20 MARKS]

5.1 Explain how earthquake can cause failure of buildings in low shallow water table sandy soils and in clay formations. [5]

5.2 A site in California is located 15km from Fault A and 40km from fault B. These two faults have maximum probable earthquake magnitudes, Mw of 6.5 and 7.8, respectively. The site is underlain by a deep deposit of soil that has a shear velocity of 610 m/s. Using the Boore et al 1993 attenuation relationship, compute. [10]

a. The peak horizontal acceleration at the ground surface for both earthquakes,

b. Select the value to be used for design.

Formula sheet

$\gamma_c = 24$ KN/m^3

$g = 9.8$ m/s^2

$\sigma_z = \sum \gamma H$

1kPa = 0.0102 kg/cm^2

$\sigma'_m = \sigma'_c - \sigma'_{Zo}$

$\sigma' = \sigma' - \mu$

$\gamma = \frac{W}{V}$

$q = \frac{P}{A} + \gamma_c D - u.$

$q = \frac{P/b}{B} + \gamma_c D - u.$

$q = \frac{P+W_f}{A} - u$

$q_a = \frac{q_{ult}}{F}$

$\Delta\sigma_Z = I_\sigma(q - \sigma'_D)$

$\sigma'_{Zf} = \sigma'_{z0} + \Delta\sigma_Z$

Rigidity factor r = 0.85

Area of circle= ∏ D^2/4

Perimeter of a circle= ∏DL

Perimeter of a rectangle= 2*(B+L)

Area of a rectangle = (B*L)

Deflection Ratio (%) = (ΔY/D) * 100%

$$\Delta X = \frac{D_L * K * W_c * r^3}{E * I + 0.061E' * r^3}$$

$$\Delta X = \frac{D_L * K * W_c}{0.149.PS + 0.061E'}$$

For Solid Pipe,

$$PS = \frac{E.I}{0.149 * r^3}$$

Wc = (Σγi. H$_i$).Do

E′ = 2Es.

For Perforated Pipe

$$P_{cr} = 2\{[E'/(1-\mu)](E.I/r^3)\}^{1/2}$$

$$W_c = \frac{(\Sigma(\gamma_i H_i)D_o}{(1-n.d/12)}$$

$I = t^3/12$

Mean diameter of the pipe: D = (Do +Di)/2 = Do – t = Di – t

The vertical pressure on Solid Pipe, (P_{tp})

$P_{tp} = \Sigma\gamma_i . H_i$

The vertical pressure on perforated Pipe, (P_{tp})

$$P_{tp} = \frac{\Sigma \gamma_i H_i}{(1-n*d/12)}$$

Table 1: Values of Bedding Constant, K

Bedding Angle θ (degree)	Bedding Constant K
0	0.110
30	0.108
45	0.105
60	0.102
90	0.096
120	0.090
180	0.083

Table 2: Approximate range of values of D_L

Variable	Range	Remarks
D_L	1.5 – 2.5	If the soil is not compacted, then the higher value should be used.
	1.0	When deflection calculation are based on prism load.

Table 3: Allowable Deflection Ratio of Polyethylene Pipe

SDR	Allowable Deflection Ratio (%)
11	2.7
13.5	3.4
15.5	3.9
17	4.2
19	4.7
21	5.2
26	6.5
32.5	8.1

Table 4: ELASTIC SOIL PARAMETER (Es) SELIG (1990)

Soil type	Stress level		85% standard Density			95% standard density		
	psi	kPa	psi	MPa	v_s	psi	MPa	v_s
SW,	1	7	1300	9	0.26	1600	11	0.40
SP	5	35	2100	14	0.21	4100	28	0.29
GW	10	70	2600	18	0.19	6000	41	0.24
GP	20	140	3300	23	0.19	8600	59	0.23
	40	280	4100	28	0.23	13000	90	0.25
	60	420	4700	32	0.28	16000	110	0.29
GM,SM,	1	7	600	4	0.25	1800	12	0.34
ML and	5	35	700	5	0.24	2500	17	0.29
GC,SC with <	10	70	800	6	0.23	2900	20	0.27
20%	20	140	850	6	0.30	3200	22	0.29
fines	40	280	900	6	0.38	3700	25	0.32
	60	420	1000	7	0.41	4100	28	0.35
CL,MH	1	7	100	1	0.33	400	3	0.42
GC,SC	5	35	250	2	0.29	800	6	0.35
	10	70	400	3	0.28	1100	8	0.32
	20	140	600	4	0.25	1300	9	0.30
	40	280	700	5	0.35	1400	10	0.35
	60	420	800	6	0.40	1500	10	0.38

Table 5: E′ for degree of compaction of bedding.

Soil type pipe bedding material (USC)	dumped	Slight, <85%,proctor <40%, relative density	moderate, 85 – 95%,proctor 40 - 70%, relative density	high, >95%,proctor >70%, relative density
Fine grained soils (LL <50). Soil with medium to high plasticity. CL,ML,CH-MH	No data available, use E′ = 0			
Fine grained soils (LL <50). Soil with medium to no plasticity. CL,ML,ML-CL, with more than 25% coarse grained particles	345 kN/m^2	1350	2760	6900
Fine grained soils (LL <50). Soil with medium to no plasticity. CL,ML,ML-CL, with more than 25% coarse grained particles Coarse grained soils with fines GM, GP, SM,SC contain more than 12% fines.	690	2760	6900	13800
Coarse grained soils with little or no fines. GW, GP, SW, SP contain less than 12% fines	1380	6900	13800	20700
Crushed rock	6900	20700	20700	20700
Accuracy in terms of % deflection	±2	±2	±1	±0.5

Table 6: Bearing capacity factors for Terzaghi's equations

ϕ' (deg)	N_c	N_q	N_γ	ϕ' (deg)	N_c	N_q	N_γ
0	5.7	1.0	0.0	21	18.9	8.3	5.1
1	6.0	1.1	0.1	22	20.3	9.2	5.9
2	6.3	1.2	0.1	23	21.7	10.2	6.8
3	6.6	1.3	0.2	24	23.4	11.4	7.9
4	7.0	1.5	0.3	25	25.1	12.7	9.2
5	7.3	1.6	0.4	26	27.1	14.2	10.7
6	7.7	1.8	0.5	27	29.2	15.9	12.5
7	8.2	2.0	0.6	28	31.6	17.8	14.6
8	8.6	2.2	0.7	29	34.2	20.0	17.1
9	9.1	2.4	0.9	30	37.2	22.5	20.1
10	9.6	2.7	1.0	31	40.4	25.3	23.7
11	10.2	3.0	1.2	32	44.0	28.5	28.0
12	10.8	3.3	1.4	33	48.1	32.2	33.3
13	11.4	3.6	1.6	34	52.6	36.5	39.6
14	12.1	4.0	1.9	35	57.8	41.4	47.3
15	12.9	4.4	2.2	36	63.5	47.2	56.7
16	13.7	4.9	2.5	37	70.1	53.8	68.1
17	14.6	5.5	2.9	38	77.5	61.5	82.3
18	15.5	6.0	3.3	39	86.0	70.6	99.8
19	16.6	6.7	3.8	40	95.7	81.3	121.5
20	17.7	7.4	4.4	41	106.8	93.8	148.5

Case 1: groundwater table is at or above base of footing ($D_w \leq D$)

Case 2: groundwater table is below the base of footing, but still within the potential shear zone, assumed to have a thickness of B, below the footing ($D < D_w < D+B$)

Case 3: groundwater table is below the potential shear zone below the footing ($D+B \leq D_w$)

Case I

$$\gamma' = \gamma_b = \gamma - \gamma_w$$

Case II

$$\gamma' = \gamma - \gamma_w[1 - \left(\frac{D_w - D}{B}\right)]$$

Case III

$$\gamma' = \gamma$$

Continuous footings: $q_{ult} = c'N_c + \sigma'_D N_q + 0.5\gamma' B N_\gamma$

For square footings: $q_{ult} = 1.3c'N_c + \sigma'_D N_q + 0.4\gamma' B N_\gamma$

For circular footings: $q_{ult} = 1.3c'N_c + \sigma'_D N_q + 0.3\gamma' B N_\gamma$

For circular loaded areas

$$I_{\sigma} = 1 - \left(\frac{1}{1+(B/2z_f)^2}\right)^{1.5}$$

For square loaded areas,

$$I_{\sigma} = 1 - \left(\frac{1}{1+(B/2z_f)^2}\right)^{1.76}$$

For continuous loaded areas (also known as strip loads) of width B and infinite length,

$$I_{\sigma} = 1 - \left(\frac{1}{1+(B/2z_f)^{1.38}}\right)^{2.60}$$

For rectangular loaded areas of width B and length L,

$$I_{\sigma} = 1 - \left(\frac{1}{1+(B/2z_f)^{1.38+0.62B/L}}\right)^{2.60-0.84B/L}$$

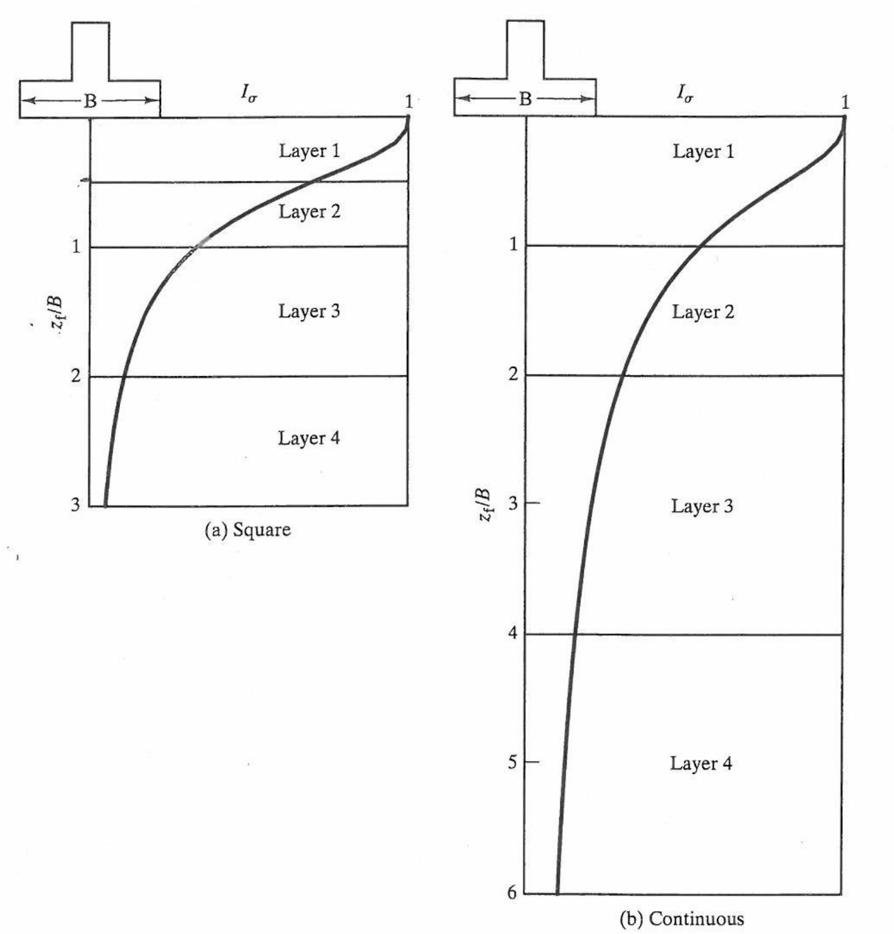


Figure 1: Guidelines for selecting layer thicknesses in hand settlement computations for uniform soils

For Normally consolidated soils

$$\delta_{C,ult} = \sum \frac{C_c}{1+e_0} H \log(\frac{\sigma'_{zf}}{\sigma'_{z0}})$$

For overconsolidated soils case I $(\sigma'_{z0} < \sigma'_{zf} \leq \sigma'_c)$

$$\delta_{C,ult} = \sum \frac{C_r}{1+e_0} H \log(\frac{\sigma'_{zf}}{\sigma'_{z0}})$$

For overconsolidated case soils II $(\sigma'_{z0} < \sigma'_c \leq \sigma'_{zf})$

$$\delta_{C,ult} = \sum [\frac{C_r}{1+e_0} H \log(\frac{\sigma'_c}{\sigma'_{z0}}) + \frac{C_c}{1+e_0} H \log(\frac{\sigma'_{zf}}{\sigma'_c})]$$

$$\delta = C_1 C_2 C_3 (q - \sigma'_D) \sum \frac{I_\varepsilon H}{E_S}$$

$$C_1 = 1 - 0.5(\frac{\sigma'_D}{q - \sigma'_D})$$

$$C_2 = 1 \; for \; t < 1$$

$$C_2 = 1 + 0.2 \log(\frac{t}{0.1}) \; for \; t \geq 1$$

$$C_3 = 1 \quad for \; square \; footings \; and \; 0.73 \; for \; continuous \; footings$$

$$I_{\varepsilon P} = 0.5 + 0.1 \sqrt{\frac{q - \sigma'_D}{\sigma'_{zp}}}$$

σ_{zp}' = vertical effective stress at depth of the peak strain influence factor (for square footings compute σ_{zp}' at a depth of D+B/2; for continuous footing compute at D+B)

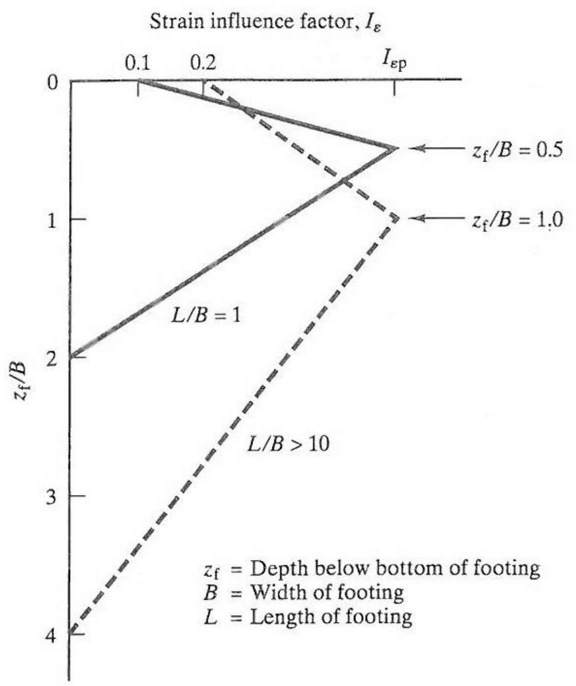


FIGURE 15.15 Distribution of strain influence factor with depth for square and continuous footings. (Adapted from Schmertmann et al., 1978, used with permission of ASCE.)

Figure 2

$E_s = 2.5q_c$ for clean sands.

$= 1.5q_c$ for clayey or silty sands.

End bearing: $q = 9\ C_u$

$S_u = Q_u/2$

Skin friction in clays: $f_{ult} = \alpha \times S_u$

Skin friction in clays (driven piles): $f = \alpha \times C_u$

f = unit skin friction; C_u = cohesion

$\alpha = 1.0$ for clays with $C_u < 25\ kN/m^2$

$= 0.5$ for clays with $C_u > 70\ kN/m^2$

Table 11.7 Bearing Capacity Factors N^*_σ Based on the Theory of Expansion of Cavities

	I_{rr}									
ϕ'	10	20	40	60	80	100	200	300	400	500
25	12.12	15.95	20.98	24.64	27.61	30.16	39.70	46.61	52.24	57.06
26	13.18	17.47	23.15	27.30	30.69	33.60	44.53	52.51	59.02	64.62
27	14.33	19.12	25.52	30.21	34.06	37.37	49.88	59.05	66.56	73.04
28	15.57	20.91	28.10	33.40	37.75	41.51	55.77	66.29	74.93	82.40
29	16.90	22.85	30.90	36.87	41.79	46.05	62.27	74.30	84.21	92.80
30	18.24	24.95	33.95	40.66	46.21	51.02	69.43	83.14	94.48	104.33
31	19.88	27.22	37.27	44.79	51.03	56.46	77.31	92.90	105.84	117.11
32	21.55	29.68	40.88	49.30	56.30	62.41	85.96	103.66	118.39	131.24
33	23.34	32.34	44.80	54.20	62.05	68.92	95.46	115.51	132.24	146.87
34	25.28	35.21	49.05	59.54	68.33	76.02	105.90	128.55	147.51	164.12
35	27.36	38.32	53.67	65.36	75.17	83.78	117.33	142.89	164.33	183.16
36	29.60	41.68	58.68	71.69	82.62	92.24	129.87	158.65	182.85	204.14
37	32.02	45.31	64.13	78.57	90.75	101.48	143.61	175.95	203.23	227.26
38	34.63	49.24	70.03	86.05	99.60	111.56	158.65	194.94	225.62	252.71
39	37.44	53.50	76.45	94.20	109.24	122.54	175.11	215.78	250.23	280.71
40	40.47	58.10	83.40	103.05	119.74	134.52	193.13	238.62	277.26	311.50
41	43.74	63.07	90.96	112.68	131.18	147.59	212.84	263.67	306.94	345.34
42	47.27	68.46	99.16	123.16	143.64	161.83	234.40	291.13	339.52	382.53
43	51.08	74.30	108.08	134.56	157.21	177.36	257.99	321.22	375.28	423.39
44	55.20	80.62	117.76	146.97	172.00	194.31	283.80	354.20	414.51	468.28
45	59.66	87.48	128.28	160.48	188.12	212.79	312.03	390.35	457.57	517.58

Table 11.5 Interpolated Values of N_q^* Based on Meyerhof's Theory

Soil friction angle, ϕ (deg)	N_q^*
20	12.4
21	13.8
22	15.5
23	17.9
24	21.4
25	26.0
26	29.5
27	34.0
28	39.7
29	46.5
30	56.7
31	68.2
32	81.0
33	96.0
34	115.0
35	143.0
36	168.0
37	194.0
38	231.0
39	276.0
40	346.0
41	420.0
42	525.0
43	650.0
44	780.0
45	930.0

$$Q_p = A_P q' N_q^* \leq \; A_p(0.5 p_a N_q^* tan\emptyset')$$

$$Q_p = A_P q_P = A_p \sigma'_O N_\sigma^*$$

$$Q_p = A_P q' N_q^*$$

$$I_{rr} = \frac{I_r}{1+I_{r\Delta}} \qquad N_\sigma^* = \frac{3N_q}{(1+2K_o)}$$

$$\sigma'_o = \left[\frac{1+2(1-sin\emptyset')}{3}\right] q' \qquad I_r = \frac{E_s}{2(1+\mu_s)q' tan\emptyset'} = \frac{G_s}{q' tan\emptyset'}$$

$$m = \frac{E_s}{(P_a)}$$, $P_a = 100\ kN/m^2$

m = 100 – 200 (loose) / 200 – 500 medium dense / 500 – 1000 dense/

$$\Delta = 0.005\left(1 - \frac{\emptyset' - 25}{20}\right)\frac{q'}{P_a}$$

$$\mu_S = 0.1 + 0.3\left(\frac{\emptyset' - 25}{20}\right)(for\ 25° \leq \emptyset' \leq 45°)$$

Skin friction in clays (Bored piles): $f = \alpha \times C_u$

f = unit skin friction; C_u = cohesion

$\alpha = 0.7$ for clays with $C_u < 25$ kN/m^2

$= 0.35$ for clays with $C_u > 70$ kN/m^2

Skin friction in clays: $f_{ult} = \beta \times \sigma'$

Skin friction in clays (KolK and Van der Velde): $f_{ult} = \alpha \times S_u$

Table 5.2 Skin friction factor

S_u/σ'	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4
α	0.95	0.77	0.7	0.65	0.62	0.60	0.56	0.55	0.53	0.52	0.50	0.49	0.48
S_u/σ'	1.5	1.6	1.7	1.8	1.9	2.0	2.1	2.2	2.3	2.4	2.5	3.0	4.0
α	0.47	0.42	0.41	0.41	0.42	0.41	0.41	0.40	0.40	0.40	0.40	0.39	0.39

$f = \alpha \cdot C_u \cdot A_p$

S = Skin friction ; C_u = cohesion; A_p = perimeter surface area of the pile

Table 5.1 α vs cohesion

Pile Type	**Soil consistency**	**Cohesion Range (kN/m^2)**	**α**
Timber and concrete piles	Very soft	0-12	0-1.0
	soft	12-24	1.0-0.96
	Medium stiff	24-48	0.96-0.75
	stiff	48-96	0.75-0.48
	Very stiff	96-192	0.48-0.33
Steel piles	Very soft	0-12	0.0-1.0
	soft	12-24	1.0-0.92
	Medium stiff	24-48	0.92-0.70
	stiff	48-96	0.70-0.36
	Very stiff	96-192	0.36-0.19

Ultimate pile capacity = ultimate end bearing capacity + ultimate skin friction

Allowable pile capacity = ultimate pile cpacity/FOS

Pile group capacity (based on capacity of individual pile) = efficiency of the pile group x single pile capacity x number of piles

Pile group capacity (based on block group failure) = skin friction of the group + End bearing of the group

Skin friction of the group= Perimeter of the group of pile x Length of pile x Cu

*End bearing of the group = 9 Cu (L*W)*

$$Efficiency\ \eta_g = 1 - \frac{\emptyset}{90^0}\{\frac{m(n-1)+n(m-1)}{mn}\}$$

$$\emptyset = tan^{-1}(d/s)$$

Pile group efficiency for clayey soils.

Pile spacing (centre to centre)	Group efficiency
3D	0.67
4D	0.78
5D	0.89
6D or more	1.00
D = diameter of pile	

Pile group efficiency for sandy soils.

Pile spacing (centre to centre)	Group efficiency
3D	0.67
4D	0.74
5D	0.80
6D	0.87
7D	0.93
8D	1.00

$$\sigma_1 = \frac{R_1}{A} = \frac{C}{nA} \pm \frac{C.e_{x}.\alpha_x}{I_x} \pm \frac{C.e_y.\alpha_y}{I_y}$$

Moment of inertia I = A * r^2 (r = distance to the pile from the axis)

$$R = \sqrt{d^2 + z_i^2}$$

$$\log_{max} a = 0.038 + 0.216(M_w - 6) - 0.777 \log R + 0.158G_B + 0.254G_C$$

$$(\frac{\tau_{c\gamma c}}{\sigma'_{zo}})_{eqk} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{zo}}{\sigma'_{zo}} \gamma_d$$

$$FS = \frac{(\frac{\tau_{c\gamma c}}{\sigma'_{zo}})_{eqk}}{(\frac{\tau_{c\gamma c}}{\sigma'_{zo}})_{m7}}$$