UNIVERSITY OF RUHUNA

Faculty of Engineering

End-Semester 7 Examination in Engineering: August 2018

Module Number: CE7305 **Module Name: Geotechnical Engineering Design**

[Three Hours]

[Answer **all questions**, each question carries **TWELVE** marks]

Q1. It is proposed to construct abutment A1 of river bridge at EK0+243~EK0+331 using spun concrete piles (pile group) as Spun Concrete Piled Embankment (SCPE) is adopted as a ground improvement technique from chainage EK0+200 to EK0+243 in the proposed Southern Expressway Extension project from Matara to Beliatta section. Two numbers of boreholes and 5 numbers of Cone Penetration Tests (CPT) had been conducted at the site in order to identify the subsurface soil profile as shown in Figure Q1.1. The water table was found to be at the existing ground surface. A series of laboratory tests were conducted for the soil samples obtained from each layer during the site investigations and results are presented in Table Q1.1.

By considering the subsurface soil profile, Geotechnical Engineer has decided to drive 0.5 m diameter precast spun concrete piles up to the bed rock (20.0 m from the existing ground surface). The pile group consists of 8 x 6 numbers of piles at 2.4 m center-to-center spacing as shown in Figure Q1.2. Based on the structural analysis, maximum working load due to bridge abutment on the pile group was found to be 32,500 kN.

Following equations with general notations may be useful in the calculations.

$$\eta = 1 - \frac{\theta}{90}\left[\frac{(n-1)m + (m-1)n}{mn}\right]$$

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

$$Q_p = A_p q' N_q^* \leq A_p q_1$$

$$q_1 = 50 N_q^* tan\varphi$$

$$q_p = q_u\left(N_\phi + 1\right)$$

where $N_\phi = \tan^2(45 + \phi/2)$

$$s_1 = \frac{(Q_{wp} + \theta Q_{ws})L}{A_p E_p}$$

$$\theta = 0.6$$

$$s_2 = \frac{q_{wp} D (1 - \mu_s^2) I_{wp}}{E_s}$$

$$I_{wp} = 0.85$$

$$s_3 = \left(\frac{Q_{ws}}{pL}\right)\frac{D}{E_s}(1 - \mu_s^2) I_{ws}$$

$$I_{ws} = 2 + 0.35\sqrt{\frac{L}{D}}$$

Figure Q1.3, Figure Q1.4 and Figure Q1.5 may also be useful in the calculations.

a) Briefly explain two situations where negative skin friction would develop on piles.

[1.0 Marks]

b) If a 1.0 m height soil fill is placed over the existing ground surface to support the pile driving machine, what would be the expected negative skin frictional force on a single pile? Unit weight of the fill material can be taken as 20.0 kN/m³.

[2.0 Marks]

c) What would be the expected allowable carrying capacity of a single pile taking the factor of safety as 2.5?

[5.5 Marks]

d) By assuming that the Young's modulus of concrete as 20.67×10^7 kPa, estimate the expected elastic settlement of a single pile?

[2.5 Marks]

e) Assuming that the factor of safety is 4.0, what would be the expected allowable pile group capacity?

[1.0 Marks]

Q2. Subsurface soil profile at a proposed building site with a trial footing for a concrete column is shown in Figure Q2.1. The soil profile consists of 4.0 thick medium dense sand layer followed by 5.0 m thick dense sand layer. It was revealed that the water table is at a depth of 1.5 m from the ground surface. The unit weight of medium dense sand above and below the water table are 16.0 kN/m³ and 17.5 kN/m³, respectively. The internal friction angle of medium dense sand in terms of effective stress is 30°. The unit weight of water can be taken as 9.81 kN/m³. The column load is applied at an eccentrically as shown in Figure Q2.2.

Following equations with usual notations may useful in the calculations.

when $\frac{e_L}{L} \geq \frac{1}{6}$ and $\frac{e_B}{B} \geq \frac{1}{6}$, effective area $A' = \frac{1}{2} B_1 L_1$

where $B_1 = B\left(1.5 - \frac{3e_B}{B}\right)$

$$L_1 = L\left(1.5 - \frac{3e_L}{L}\right)$$

$$q_u = cN_c F_{cs} F_{cd} F_{ci} + qN_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

$$F_{cs} = 1 + \frac{B}{L}\frac{N_q}{N_c}, \quad F_{qs} = 1 + \frac{B}{L}\tan\phi \quad \text{and} \quad F_{\gamma s} = 1 - 0.4\frac{B}{L}$$

when $\frac{D_f}{B} \leq 1$ $\quad F_{cd} = 1 + 0.4\frac{D_f}{B}$, $F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \frac{D_f}{B}$ and $F_{\gamma d} = 1$

when $\frac{D_f}{B} > 1$ $\quad F_{cd} = 1 + 0.4\tan^{-1}\left(\frac{D_f}{B}\right)$, $F_{qd} = 1 + 2\tan\phi(1-\sin\phi)^2 \tan^{-1}\left(\frac{D_f}{B}\right)$ and

$$F_{\gamma d} = 1$$

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta^\circ}{90^\circ}\right)^2 \quad \text{and} \quad F_{\gamma i} = \left(1 - \frac{\beta}{\phi}\right)^2$$

$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$ when $0 \leq d \leq B$

Table Q2.1 may also useful in the calculations.

a) Show that $\frac{e_L}{L} \geq \frac{1}{6}$ and $\frac{e_B}{B} \geq \frac{1}{6}$ for the given footing.

[0.5 Marks]

b) Hence, estimate the ultimate load of the given footing.

[5.5 Marks]

c) In order to verify the bearing capacity of the soil at the foundation level, it was decided to conduct a Plate Load test. Briefly describe the plate load test procedure with suitable sketches.

[2.0 Marks]

d) Plate load test results of the footing are shown in Figure Q2.3. Diameter of the plate is 0.3 m.

i) Based on plate load test results, what would be the expected allowable bearing capacity of the footing assuming that factor of safety is 3.0?

[1.5 Marks]

ii) Based on plate load test results, what would be the expected settlement of the footing?

[1.0 Marks]

iii) List three factors to be considered when conducting plate load test.

[1.5 Marks]

Q3. Figure Q3.1 depicts an abutment in an earth dam constructed for a reservoir in a rural area. The trial slip circle is also shown in the same figure. The slope of the earth dam is 1:1.5 (vertical: horizontal) and has a vertical height of 10.0 m. The saturated unit weight of the soil is 18.5 kN/m^3 and its undrained cohesion is 40 kN/m^2. Assume that soil is fully saturated. The unit weight of water can be taken as 9.81 kN/m^3.

a) If the area of the slip mass ABD (W) is 77.35 m^2 and distance from O to centroid of the slip mass (d) is 6.5 m, determine the factor of safety against immediate shear failure ignoring the tension crack.

[2.5 Marks]

b) By considering the undrained analysis, determine the depth of the tension crack(Z_0)?

[1.0 Marks]

c) If the area of the slip mass ADFE (W) is 71.64 m^2 and distance from O to centroid of the slip mass (d) is 5.86 m, determine the factor of safety against immediate shear failure allowing for the tension crack when the reservoir is empty. Assume that there is no any water in the tension crack.

[3.0 Marks]

d) If distance from O to centroid of the water mass AGD (d_w) is 2.0 m, determine the factor of safety against immediate shear failure considering the tension crack when the reservoir is full of water. Note that mass of water in the AGD block is denoted as W_w in the Figure Q3.1. Assume that tension crack is filled with water.

[3.5 Marks]

e) It was observed that there is a considerable amount of water leakage through the earth dam when the reservoir is full of water. Suggest suitable methods to rectify the water leakage.

[1.0 Marks]

f) In order to increase the capacity of the reservoir, a politician has suggested to do excavation from bottom of the existing reservoir. As a junior Geotechnical Engineer, do you agree for this decision? Justify your answer.

[1.0 Marks]

Q4. In a road widening project of a 'A' class road, due to limited Right of Way (ROW), it is necessary to make a 5.0 m height vertical cut and which is stabilized by constructing a mass concrete retaining wall as shown in Figure Q4.1. The existing ground consists of lateritic clay and material properties of lateritic clay are shown in Table Q4.1.

The allowable bearing pressure at the foundation level can be taken as 275 kPa. Based on site investigations, it was found that water table is well below the ground surface. The unit weights of water and concrete can be taken as 9.81 kN/m^3 and 24.0 kN/m^3, respectively. The variation of K_A with φ'_{design} is illustrated in Figure Q4.2.

a) Determine the design shear strength parameters according to BS8002.

[1.0 Marks]

b) What would be the expected active earth force on retaining wall according to BS8002?

[3.0 Marks]

c) Evaluate the stability of the retaining wall against sliding according to BS8002.

[3.5 Marks]

d) Evaluate the stability of the retaining wall against overturning according to BS8002.

[1.0 Marks]

e) Evaluate the stability of the retaining wall against bearing failure according to BS8002.

[2.5 Marks]

f) What would be the expected resisting force if a shear key is provided at the base of the retaining wall?

[1.0 Marks]

Q5. There is a proposal to construct an underground waste water pumping station for Colombo Municipal Council at Borella. The underground pumping station is constructed by excavating 8.0 m deep pit below the existing ground level. Since proposed development is within the highly urbanized area, this deep excavation would cause ground subsidence and could affect the adjacent structures. Therefore, the perimeter of the proposed excavation is planned to be retained by constructing diaphragm wall upto a depth of 15.0 m and, intermittently supported by a system of wales and struts. The water table is close to the existing ground surface and proposed pumping station to be kept dry by continuous pumping of water. A schematic arrangement of the proposed setup is shown in Figure Q5.1.

The subsurface soil profile at the proposed development consists of 3.0 m thick loose sand layer followed by dense sand layer. Bed rock was encountered at a depth of 20.0 m from the ground surface. Geotechnical characteristics of the sub surface soil are shown in Table Q5.1.

The variation of K_A with φ'_{design} and variation of K_P with φ'_{design} are illustrated in Figure Q4.2 and Figure Q5.2, respectively.

a) Briefly describe the construction procedure of Diaphragm walls with suitable sketches.

[1.0 Marks]

b) Assuming that the permeability coefficients of loose sand and dense sand are same, what would be the expected pore water pressure at the toe of the diaphragm wall and at a depth of 3.0 m from the ground surface according to BS8002?

[3.5 Marks]

c) Evaluate the stability of the proposed diaphragm wall according to BS8002?

[6.5 Marks]

d) If the stability is not sufficient, suggest suitable two methods to improve the stability of the proposed diaphragm wall.

[1.0 Marks]

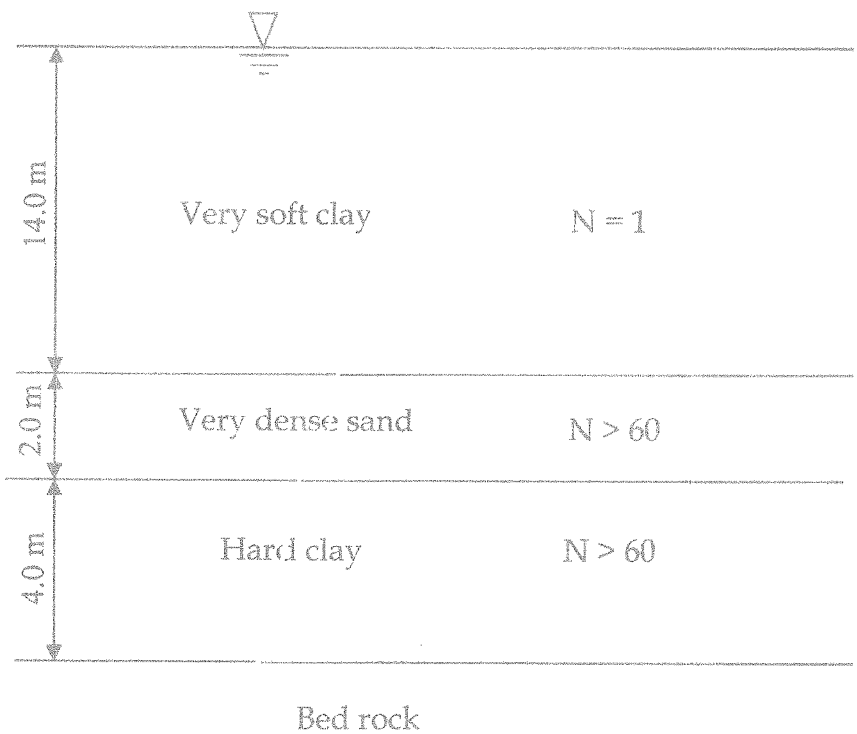

Figure Q1.1 Subsurface soil profile at abutment A1 of river bridge at EK0+243~EK0+331

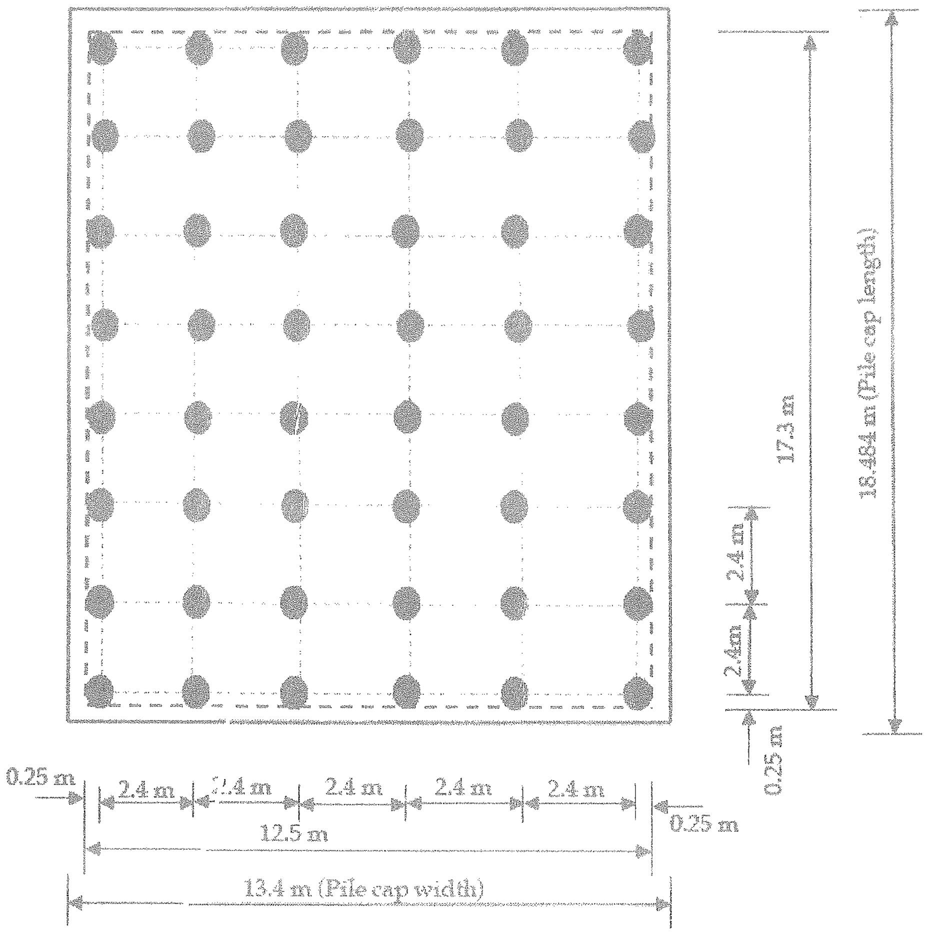

Figure Q1.2 Pile arrangement within the pile group

Table Q1.1 – Engineering properties of subsurface material

	Very soft clay	Very dense sand	Hard clay
Drained cohesion c' (kN/m^2)	0	0	10
Undrained cohesion C_u(kN/m^2)	5.0	2.0	300
Drained friction angle φ' (^{0})	18	35	30
Saturated unit weight γ_{sat} (kN/m^3)	14.0	16.0	18.0
Poisson's ratio	0.50	0.35	0.30
Young's Modulus (kN/m^2)	1,000	30,000	50,000
Unconfined Compressive strength of bed rock = 20 MPa Drained friction angle of bed rock = 40° Poisson's ratio of bed rock = 0.15 Young's Modulus of bed rock = 4200 MPa			

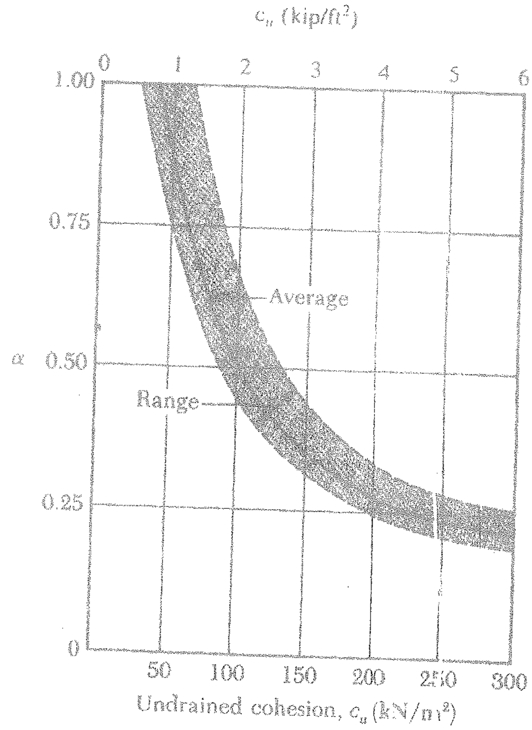

Figure Q1.3 - Variation of α with undrained cohesion of clay

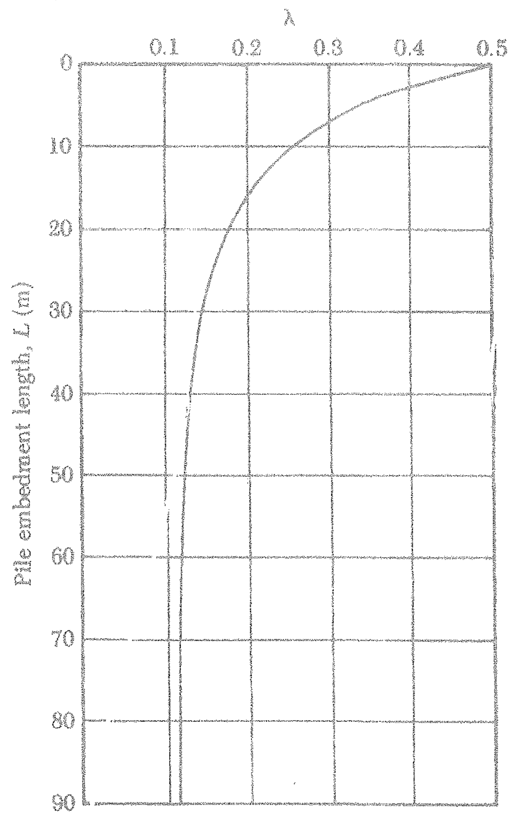

Figure Q1.4 - Variation of λ with pile embedded length

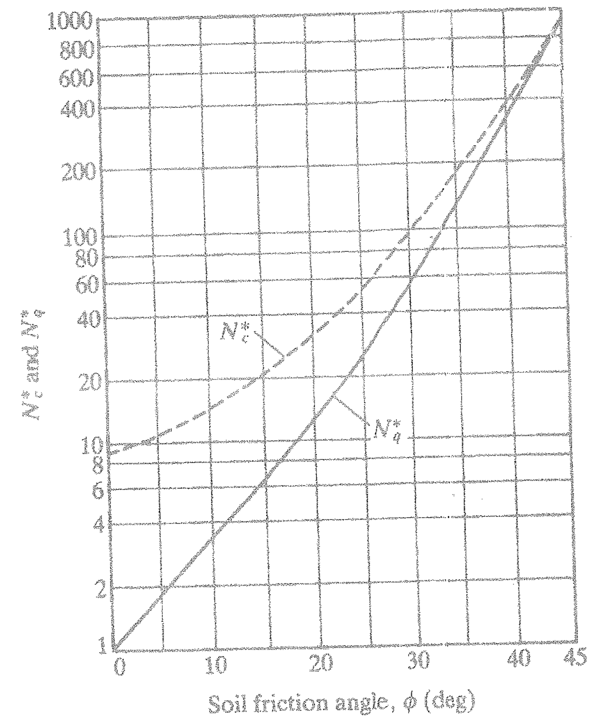

Figure Q1.5 - Variation of N_c^* and N_q^* with soil friction angle

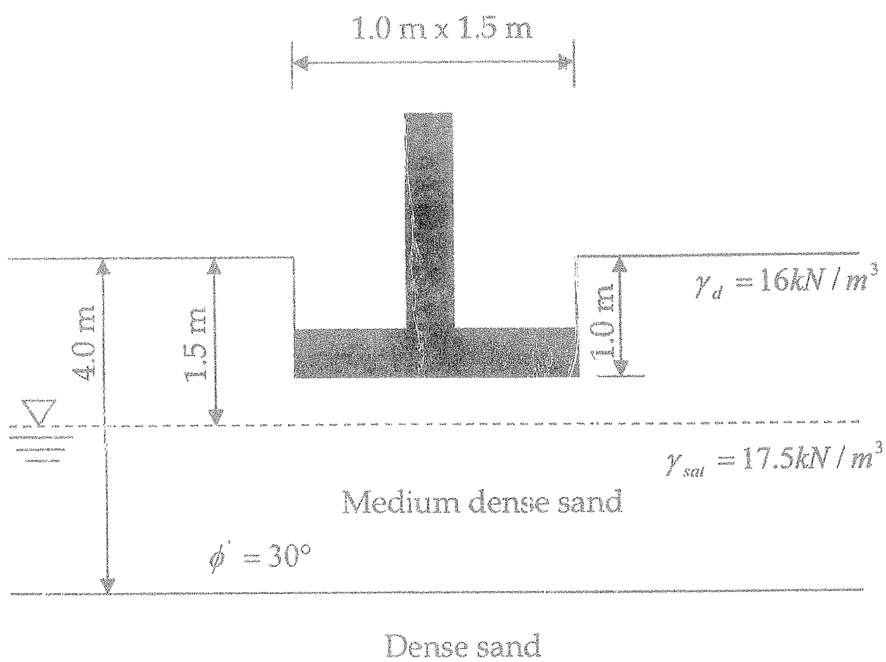

Figure Q2.1 Subsurface profile at the proposed building site

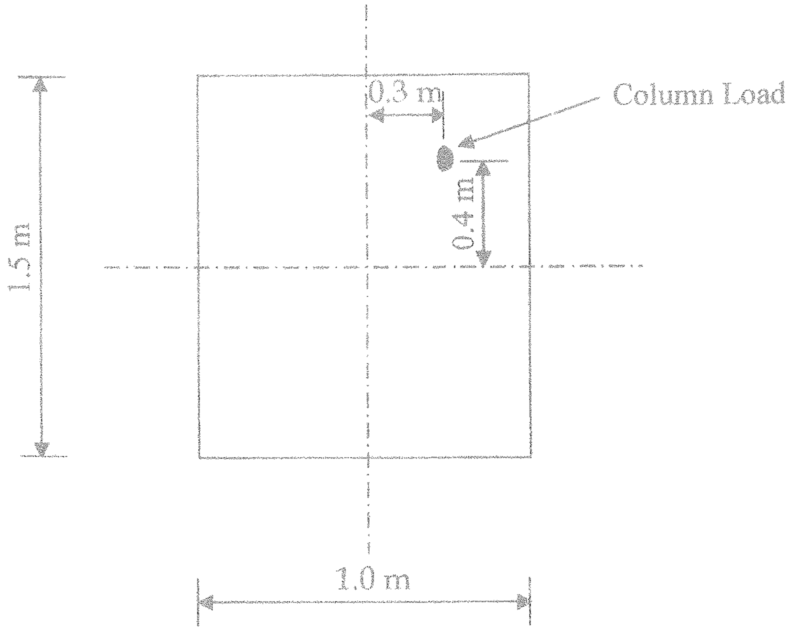

Figure Q2.2 Location of the column load with respect to footing

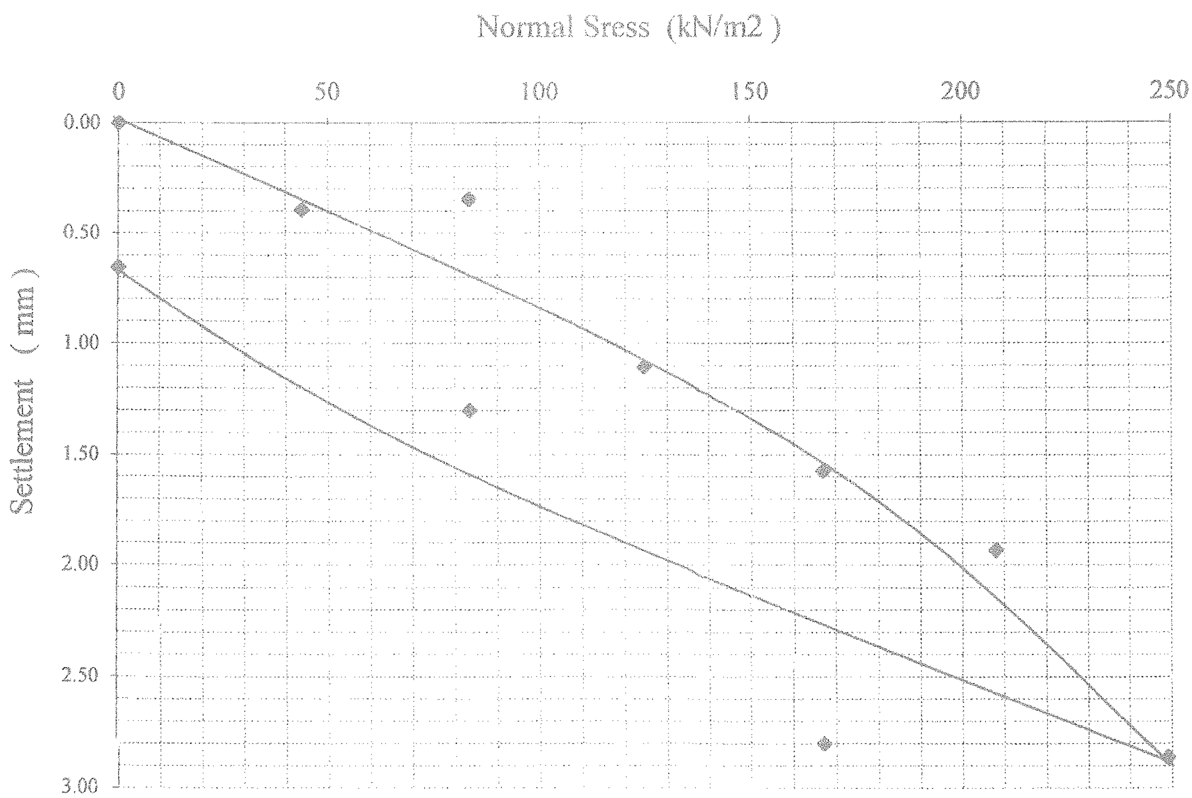

Figure Q2.3 - Plate load test results

Table Q2.1 - Bearing capacity factors

ϕ	N_c	N_q	N_γ	N_q/N_c	$\tan\phi$	ϕ	N_c	N_q	N_γ	N_q/N_c	$\tan\phi$
0	5.14	1.00	0.00	0.20	0.00	26	22.25	11.85	12.54	0.53	0.49
1	5.38	1.09	0.07	0.20	0.02	27	23.94	13.20	14.47	0.55	0.51
2	5.63	1.20	0.15	0.21	0.03	28	25.80	14.72	16.72	0.57	0.53
3	5.90	1.31	0.24	0.22	0.05	29	27.86	16.44	19.34	0.59	0.55
4	6.19	1.43	0.34	0.23	0.07	30	30.14	18.40	22.40	0.61	0.58
5	6.49	1.57	0.45	0.24	0.09	31	32.67	20.63	25.99	0.63	0.60
6	6.81	1.72	0.57	0.25	0.11	32	35.49	23.18	30.22	0.65	0.62
7	7.16	1.88	0.71	0.26	0.12	33	38.64	26.09	35.19	0.68	0.65
8	7.53	2.06	0.86	0.27	0.14	34	42.16	29.44	41.06	0.70	0.67
9	7.92	2.25	1.03	0.28	0.16	35	46.12	33.30	48.03	0.72	0.70
10	8.35	2.47	1.22	0.30	0.18	36	50.59	37.75	56.31	0.75	0.73
11	8.80	2.71	1.44	0.31	0.19	37	55.63	42.92	66.19	0.77	0.75
12	9.28	2.97	1.69	0.32	0.21	38	61.35	48.93	78.03	0.80	0.78
13	9.81	3.26	1.97	0.33	0.23	39	67.87	55.96	92.25	0.82	0.81
14	10.37	3.59	2.29	0.35	0.25	40	75.31	64.20	109.41	0.85	0.84
15	10.98	3.94	2.65	0.36	0.27	41	83.86	73.90	130.22	0.88	0.87
16	11.63	4.34	3.06	0.37	0.29	42	93.71	85.38	155.55	0.91	0.90
17	12.34	4.77	3.53	0.39	0.31	43	105.11	99.02	186.54	0.94	0.93
18	13.10	5.26	4.07	0.40	0.32	44	118.37	115.31	224.64	0.97	0.97
19	13.93	5.80	4.68	0.42	0.34	45	133.88	134.88	271.76	1.01	1.00
20	14.83	6.40	5.39	0.43	0.36	46	152.10	158.51	330.35	1.04	1.04
21	15.82	7.07	6.20	0.45	0.38	47	173.64	187.21	403.67	1.08	1.07
22	16.88	7.82	7.13	0.46	0.40	48	199.26	222.31	496.01	1.12	1.11
23	18.05	8.66	8.20	0.48	0.42	49	229.93	265.51	613.16	1.15	1.15
24	19.32	9.60	9.44	0.50	0.45	50	266.89	319.07	762.89	1.20	1.19
25	20.72	10.66	10.88	0.51	0.47						

*After Vesic (1973)

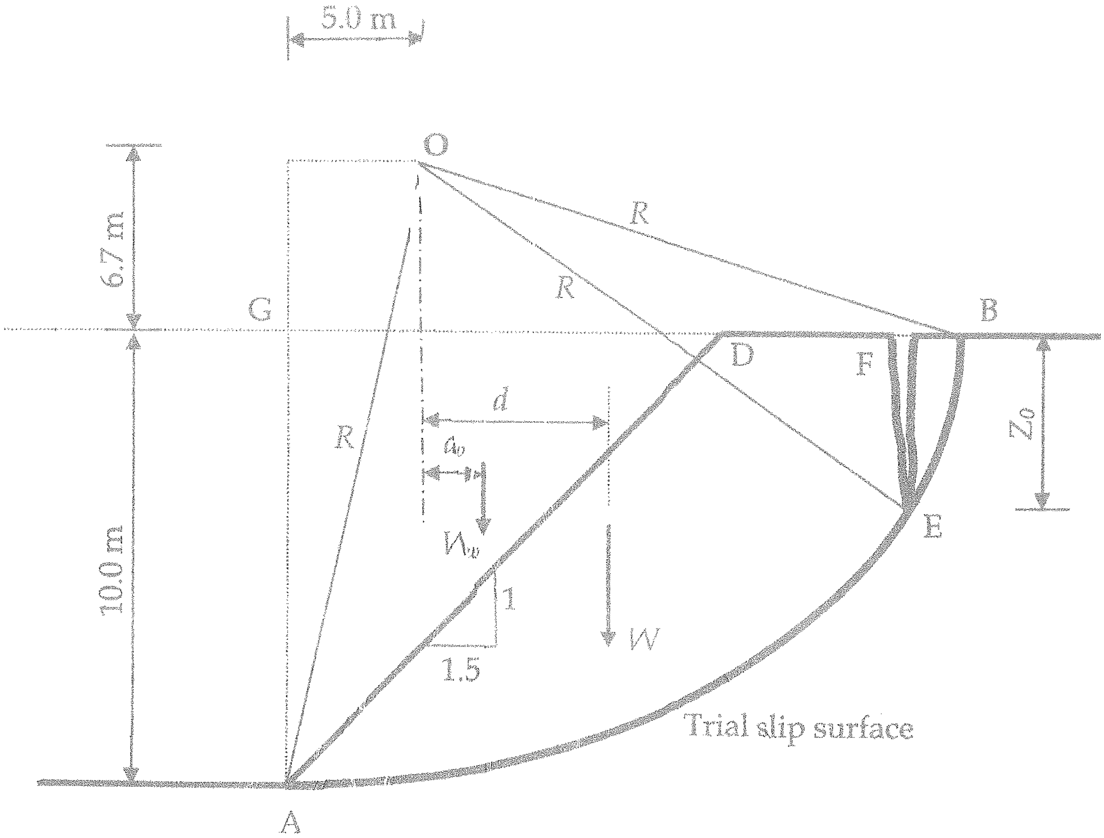

Figure Q3.1 - Circular slope failure

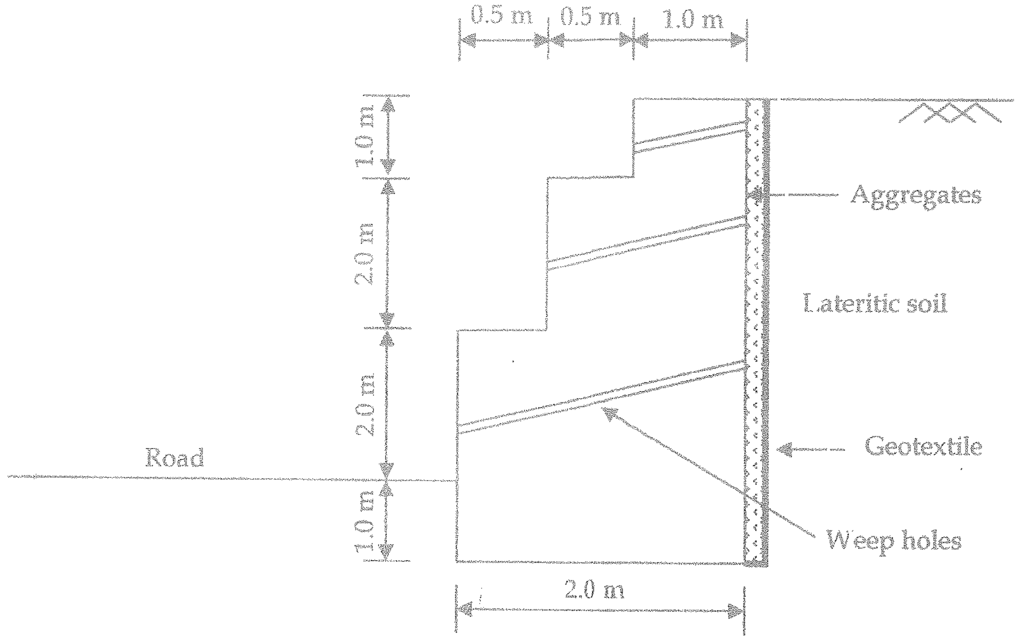

Figure Q4.1 Cross section of retaining wall

Table Q4.1 Properties of lateritic clay

Dry unit weight (γ_{dry})	16.5 kN/m³
Saturated unit weight (γ_{sat})	18.0 kN/m³
Drained friction angle (φ')	30°
Undrained friction angle (φ_u)	3°
Drained cohesion (c')	10.0 kN/m²
Undrained cohesion(c_u)	50.0 kN/m²

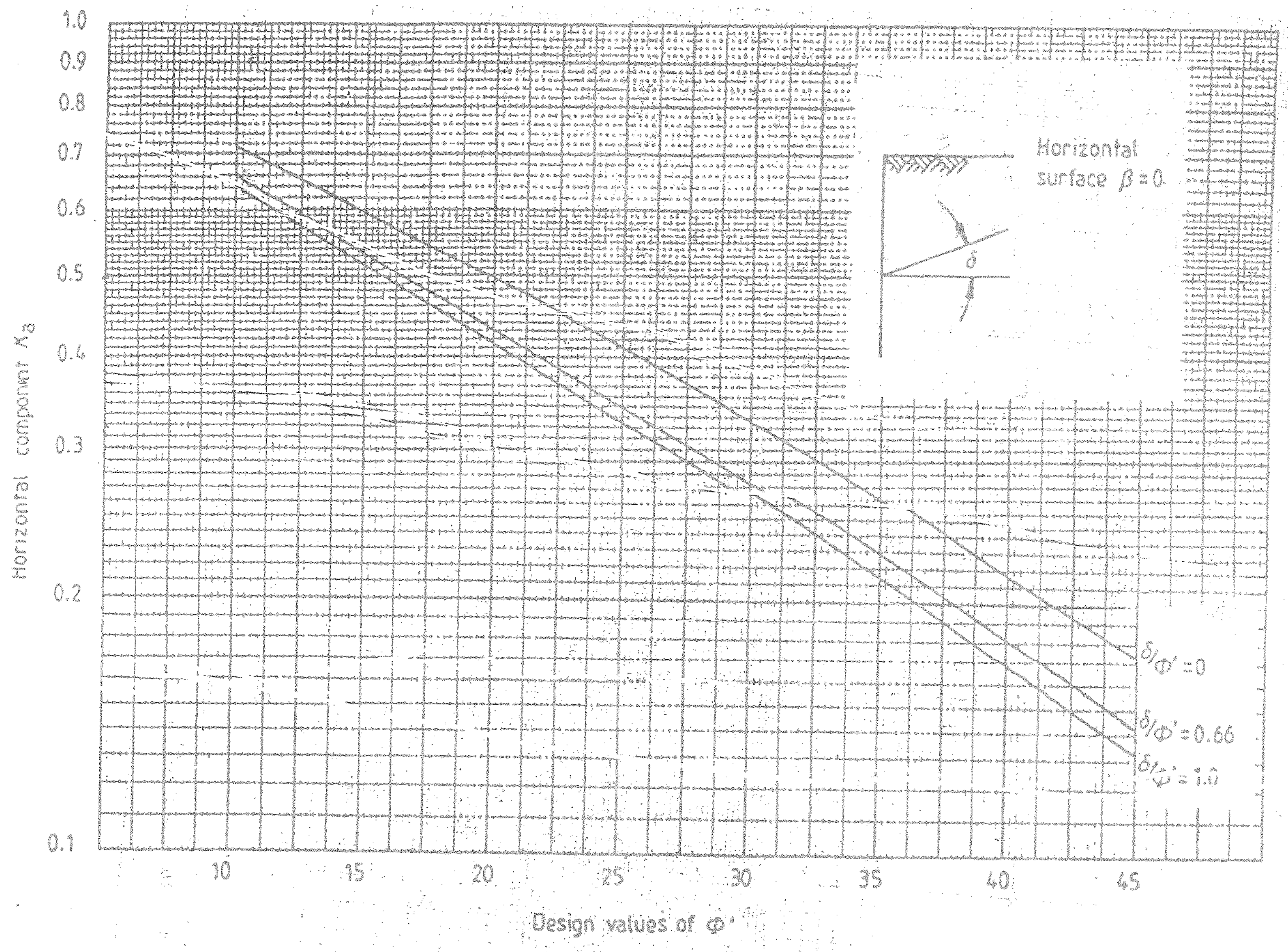

Figure Q4.2 Variation of K_A with φ'_{design}

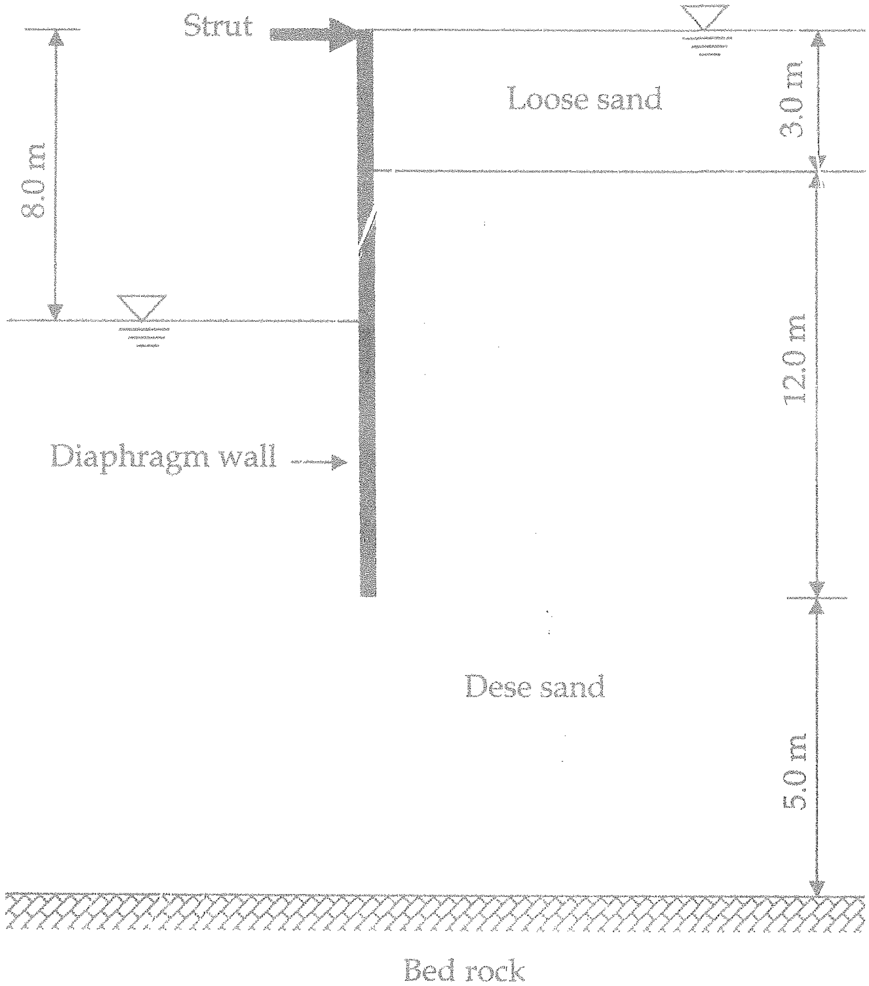

Figure Q5.1 – Subsurface soil profile at the diaphragm wall construction project

Table Q5.1 – Engineering properties of subsurface soil

	Loose sand	Dense sand
Drained cohesion c' (kN/m²)	0	0
Undrained cohesion c_u (kN/m²)	5.0	0
Drained friction angle φ' (^{0})	30	30
Saturated unit weight γ_{sat} (kN/m³)	18.0	20.0

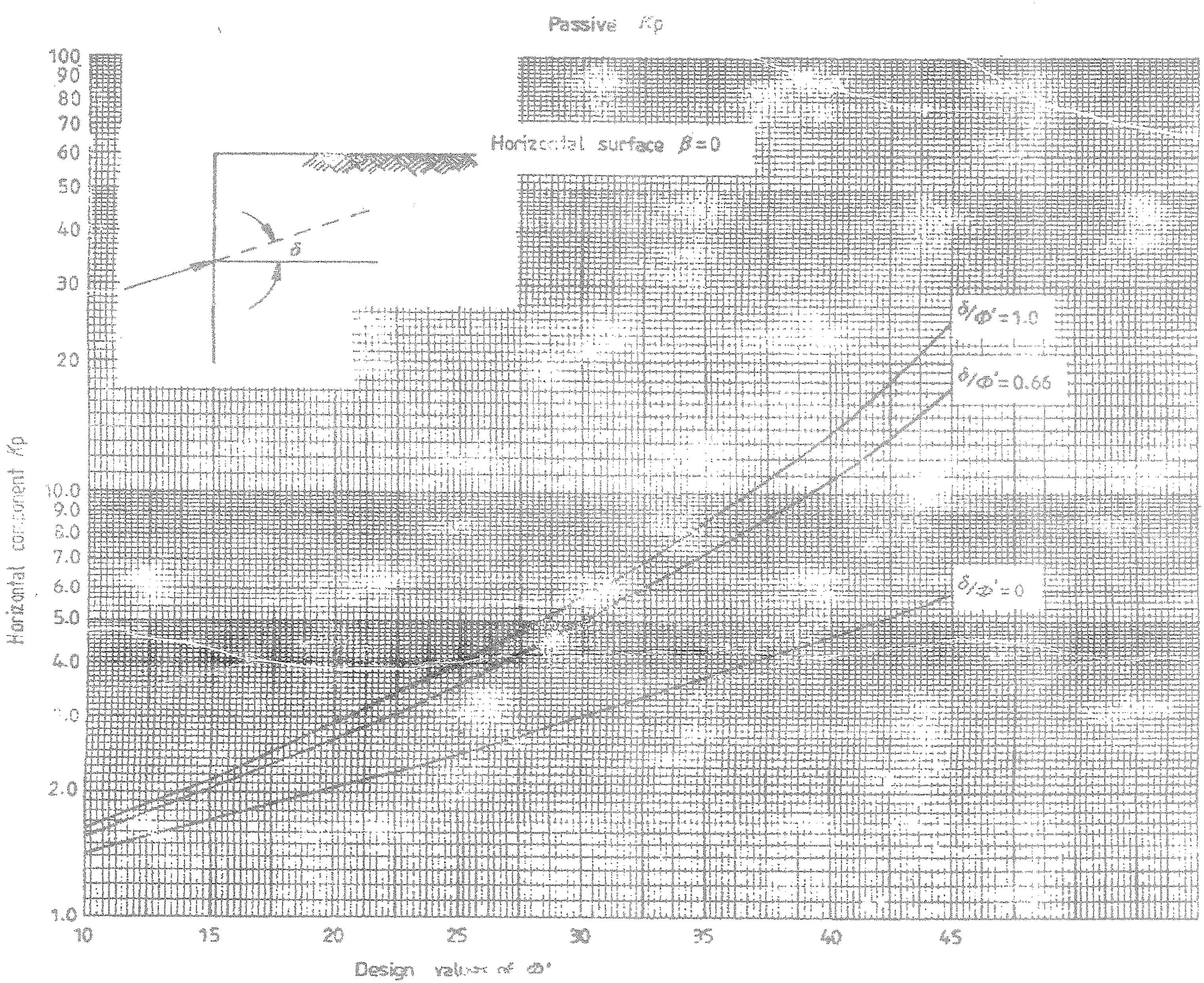

Figure Q5.2 Variation of K_P with φ'_{design}