ออกแบบเสา JETTY

เสาเข็มรับน้ำหนักแนวแกน	=	140	ตัน/ต้น
รับแรงในแนวราบ	=	14	ตัน/ต้น
รับโมเมนต์	=	10	ตัน-ม.

ใช้เข็ม 0.45x0.45 m.

Casing for bore pile	=	1.00	เมตร
Effective diameter	=	0.80	เมตร

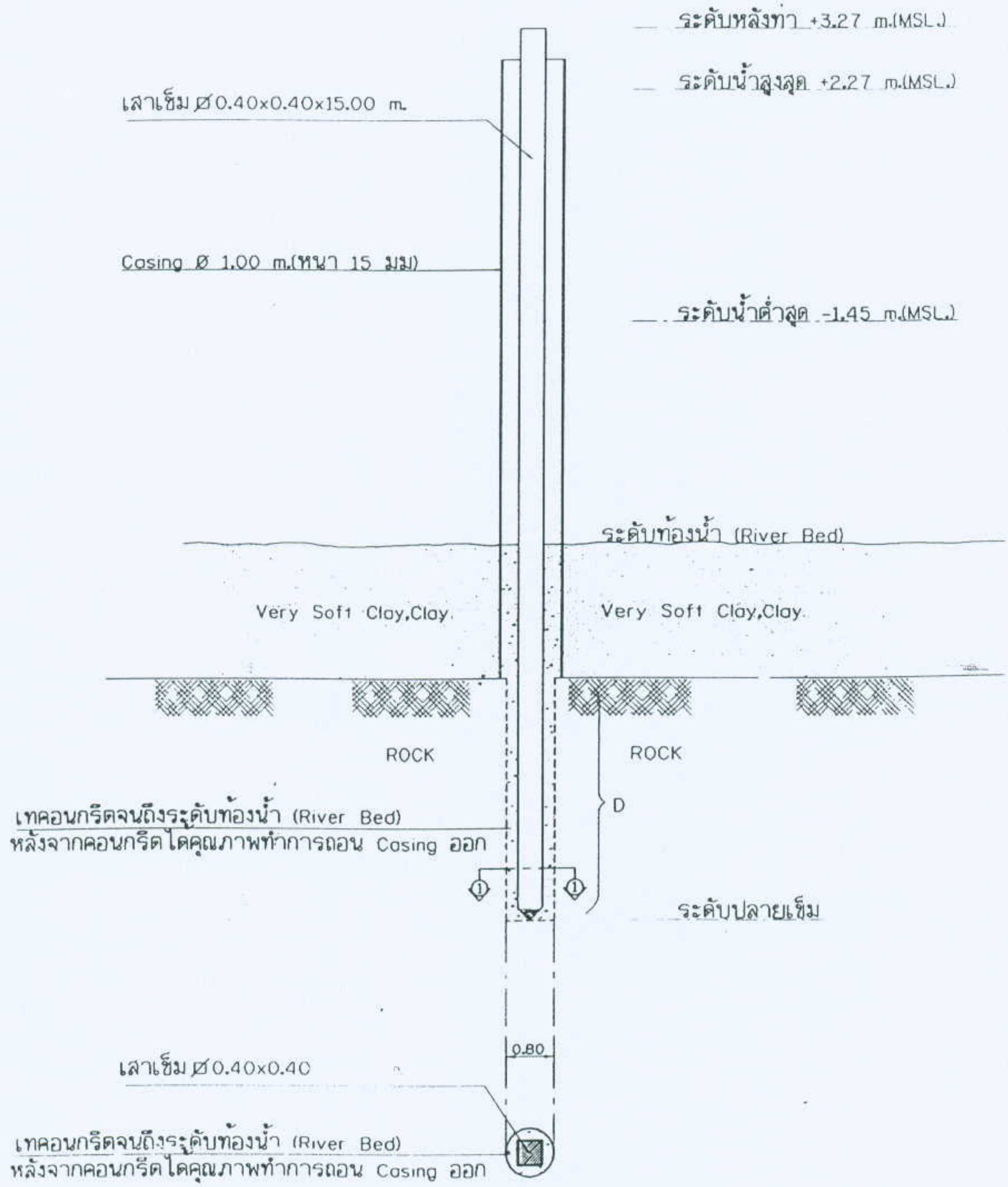
Qe = qexA ; qe = End Bearing

Qe = End Bearing Capacity of Pile

Qe = $1000 \times \pi \times 0.8^2 / 4$ = 503 ตัน/ต้น

Qa = Qe/F.S. = Allowable load Capacity of pile

= 503 / 3 = 167 ตัน มากกว่า 140 ตัน ผ่าน.



รูปตัด ① - ①

รูปแสดงการตอกเสาเข็ม คอร. ลงบนชั้นหิน

ออกแบบระยะฝังของเสาเข็ม JETTY ในชั้นหิน

Consider a pile of length L subjected to a lateral force FH and drilled shaft extending into rock .

FH	=	14	tons.
Trial L	=	4	m.
D_s	=	800	mm.

averageHorizontal soil modulus $E_s = 3.515 \times 10^6$ kN / m²

E_p	=	21×10^7	kN / m².
∅	=	44°	

Use Meyerhof 's method

Solution From Eq., relative stiffness of the shaft,

$$K_r = \frac{E_p I_p}{E_s L^4}$$

$$I_p = \frac{\pi}{64} D_s^4 = \frac{\pi}{64}\left(\frac{800}{1000}\right)^4 = 0.02 \quad m^4$$

$$K_r = \frac{(21x10^7)(0.02)}{(3.515x10^6)(4)^4} = 0.005$$

Since K_r is less than 0.01 this is a flexible drilled shaft.

From Eq.

$$\frac{L_e}{L} = 1.65\ K_r^{0.12}$$

$$L_e = (1.65)(0.005)^{0.12}(4) = 3.50 \quad m.$$

Thus, from Eq.

$$FH = 0.12\gamma D_s L_e^2 K_{br} \leq 0.4\, p_l\, D L_e$$

$$\frac{L_e}{D_s} = \frac{3.50}{0.8} = 4.375$$

From Figure 9.45, for L_e / D_s = 4.375 and $\phi 44°$, the value of $K_{br} \approx 16$, so

$$FH = (0.12)(22)(0.8)(3.50)^2(16) = 414 \quad kN$$

Check:

$$FH \quad 0.4\, p_l\, D_s L_e \quad (0.4)(40\ N_q \tan\phi) D_s L_e$$

For ϕ = 44°, N_q = 115.31 (table 3.4),

FH = (0.4)(40)(115.31)(tan 44)(0.8)(3.50) = 4989 kN

So,

FH	=	414 kN =	41.40	Tons
Fh_{allw}	=	41.40/3 =	**14**	**Tons.**
USE L	=	**4**	**m.**	

Check the allowable load-bearing capacity of drilled shaft extending into rock (4.00 m.).

FV	=	140	tons.			
L	=	4.00	m.	=	13.123	ft.
D_c	=	0.80	m.	=	2.62	ft.
q_u (rock)	=	5,000			lb / in^2.	
q_u (concrete)	=	3,983			lb / in^2.	
E_c	=	29,871,977			lb / in^2.	
RQD(rock)	=	40%				
E_{core} (rock)	=	0.50×10^6			lb / in^2.	

Step 1. From $f(lb/in^2)$ = $2.50 q_u^{0.50} \leq 0.15\, q_u$

Since q_u (concrete) < q_u (rock), use q_u(concrete). Hence

f = $2.50(3983)^{0.50}$ = 157.80 lb/in^2

Check:

= $0.15\, q_u$ = 0.15x3983 = 597.45 lb/in^2 > 157.80 lb/in^2

So, use f = 157.80 lb/in^2.

Step 2. From $Qu = \pi D_c L f = \pi x(2.62x12)x(13.123x12)x(157.80)/1000$

= 2,455 kip.

Step 3. From $S = \dfrac{(QuL)}{(AcEc)} + \dfrac{(QuI_f)}{(DsE_{mass})}$

For RQD ≈ 40 %, from Figure 10.28, the value of $E_{mass}/E_{core} \approx 0.10$, thus

E_{mass} = $0.10 E_{core}$ = $0.10 x 0.50 x 10^6$ = 50,000 lb/in^2.

E_{con}/E_{mass} = 29,871,977 / (50,000) ≈ 598

L/Ds = 13.123/2.62 5

From Figure 10.27 , for Ec/Emass = 598 and L/Ds = 5, the magnitude of I_f is about 0.25 . Hence

$$S = \frac{(2455x10^3)(13.123x12)}{(\pi/4)(2.62x12)^2(29871977)} + \frac{(2455x10^3)(0.25)}{(2.62x12)(50000)}$$

= 0.41 in. ok.

Hence; Qu = 2,455 kip.

Qall = Qu/F.S.

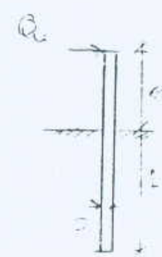
= 2455/3

= 818 kip = 371 tons > 140 tons OK.

Check the allowable moment of drilled shaft extending into rock (4.00 m.).

Moment = 10 ton-m.

Q_U = 1 ton.



Kp = $\tan^2(40+\varnothing/2)$

= $\tan^2(40+44/2)$

= 5.55

L/D = 4/0.80

= 5

e/L = 10/4

= 2.50

From Figure 9.41(a) , for L/D = 5 and e/L = 2.5, the magnitude of $\frac{Q_U}{K_p \gamma D^3}$ is about 1 . Hence

Q_U = 5.55 X 2200 X (0.80^3)

= 6252 kg.

Q_{Uallw} = 6252/3/1000

= 2 tons > 1 ton OK

ตารางที่ 1.9 คุณสมบัติทางวิศวกรรมของหิน

Type of rock	Typical unit wt., pcf	Modulus of elasticity E, ksi†	Poisson's ratio μ	Compressive strength, ksi
Basalt	178	2500–15 000	0.27–0.32	25–60
Granite	168	2000–12 000	0.26–0.30	10–40
Schist	165	1000–12 000	0.18–0.22	5–15
Limestone	165	3000–15 000	0.24–0.45	5–25
Porous limestone		500–12 000	0.35–0.45	1–5
Sandstone	145–150	500–6000	0.20–0.45	4–20
Shale	100–140	500–3000	0.25–0.45	1–6
Concrete	100–150	Variable	0.15	2–6

† Depends heavily on confining pressure and how determined. E = tangent modulus at approximately 50 percent of ultimate compression strength.

▼ TABLE 3.4 Bearing Capacity Factors[a]

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						

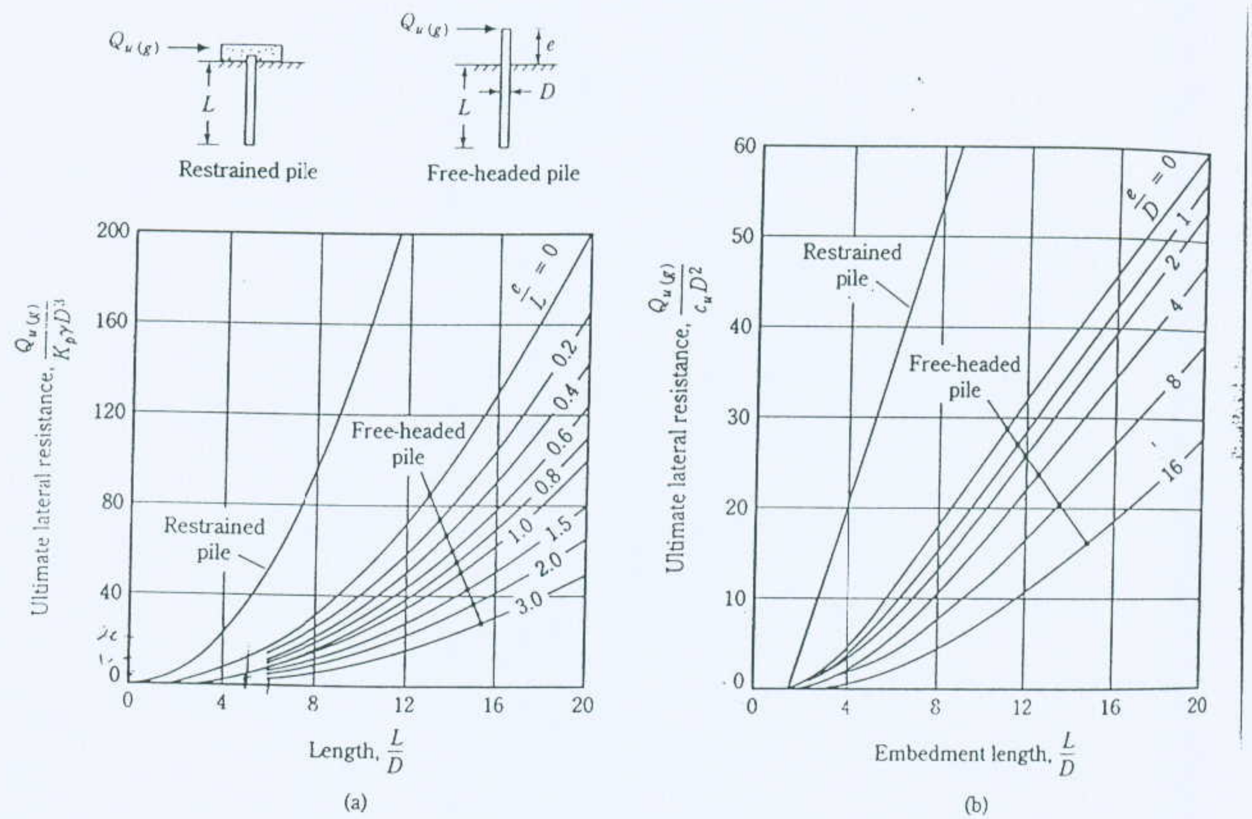
[a] After Vesic (1973)

▼ TABLE 9.3 Typical Unconfined Compressive Strength of Rocks

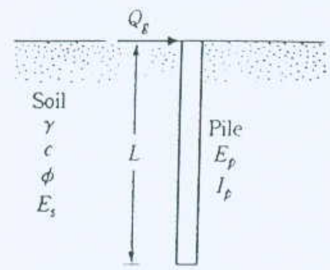
Rock type	q_u lb/in²	q_u MN/m²
Sandstone	10,000–20,000	70–140
Limestone	15,000–30,000	105–210
Shale	5,000–10,000	35–70
Granite	20,000–30,000	140–210
Marble	8,500–10,000	60–70

▼ TABLE 9.4 Typical Values of Angle of Friction, ϕ, of Rocks

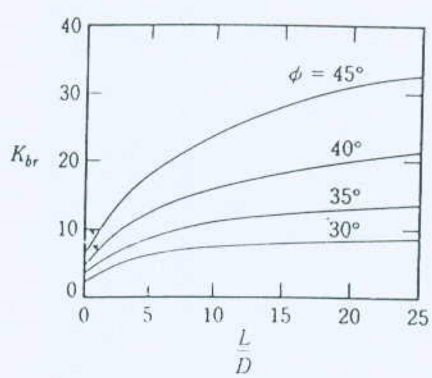
Rock type	Angle of friction, ϕ (deg)
Sandstone	27–45
Limestone	30–40
Shale	10–20
Granite	40–50
Marble	25–30



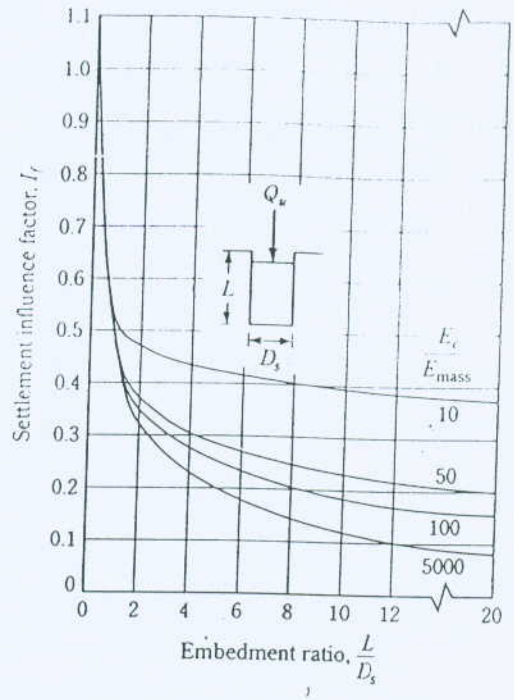
▼ FIGURE 9.41 Broms' solution for ultimate lateral resistance of short piles (a) in sand, (b) in clay



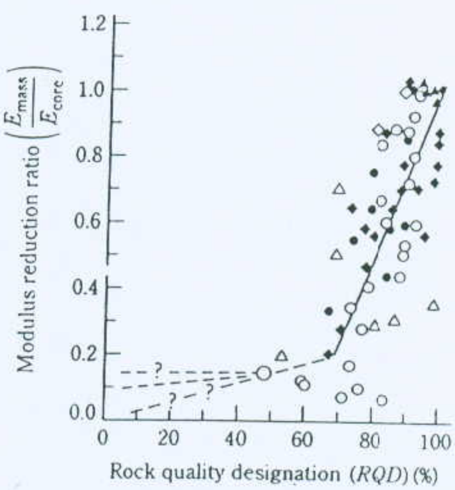
▼ FIGURE 9.44 Pile with lateral loading at ground level



▼ FIGURE 9.45 Variation of resultant net soil pressure coefficient, K_{br}



▼ **FIGURE 10.27** Variation of I_f (after Reese and O'Neill, 1989)



▼ **FIGURE 10.28** Plot of E_{mass}/E_{core} vs. RQD (after Reese and O'Neill, 1989)

ออกแบบ Cap Beam Jetty

D.L = 24,428.80 kg/m

L.L. = 17,700 kg/m.

W = 42,128.80 kg/m.

fy = 3000 ksc.

fs = 1500 ksc.

fc' = 280 ksc.

fc = 65 ksc.

n = 8.0642

j = 0.9137

k = 0.259

R = 7.6896

ขนาดคาน 0.90 x 0.80 m.

d = 0.74 m.

MR = $7.6896 \times 0.9 \times (0.74)^2$

= 37898 kg-m.

M' = $WL^2/16$

= $42{,}128.80(2.70)^2/16$ = 19,195 kg-m.

As' = 19,195/(1500x0.9137x0.74)

= 18.93 cm^2.

Use 4 - DB 25 mm. ; As = 15.412 $cm.^2$

7 - DB 28 mm. ; As = 33.838 $cm.^2$

Total As = 49.25 $cm.^2$

M = $WL^2/16$

= $42{,}128.80(2.70)^2/11$ = 27,920 kg-m.

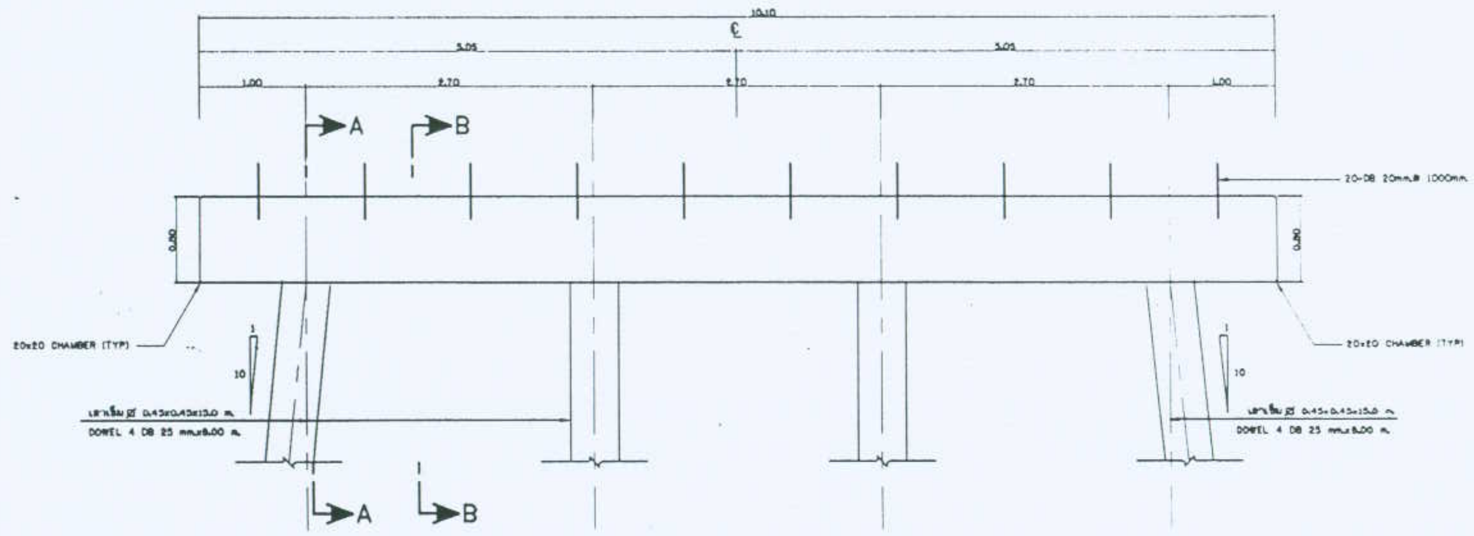
As = 27,920/(1500x0.9137x0.74)

= 27.55 cm^2.

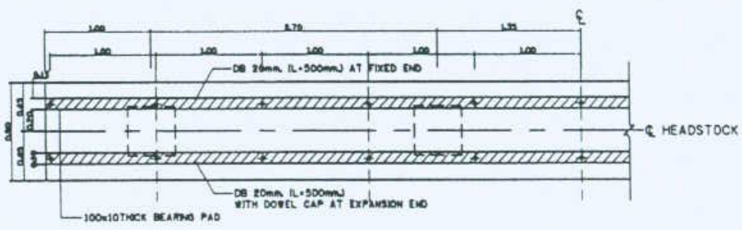
Use 4 - DB 25 mm. ; As = 15.412 $cm.^2$

7 - DB 28 mm. ; As = 33.838 $cm.^2$

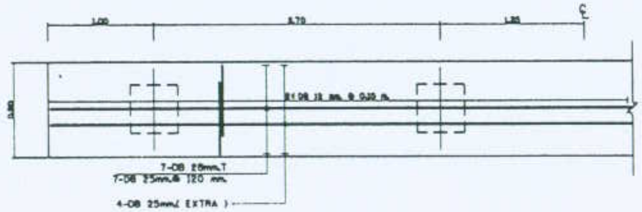
Total As = 49.25 $cm.^2$



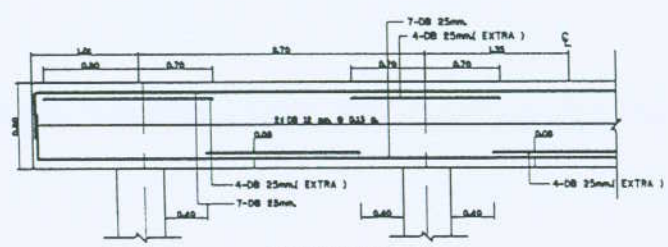
FRONT ELEVATION
SCALE 1 : 25



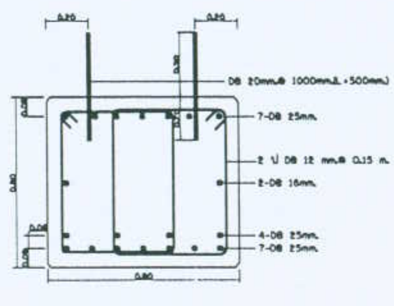
HALF PLAN PIER P(T)



HALF SECTION PLAN

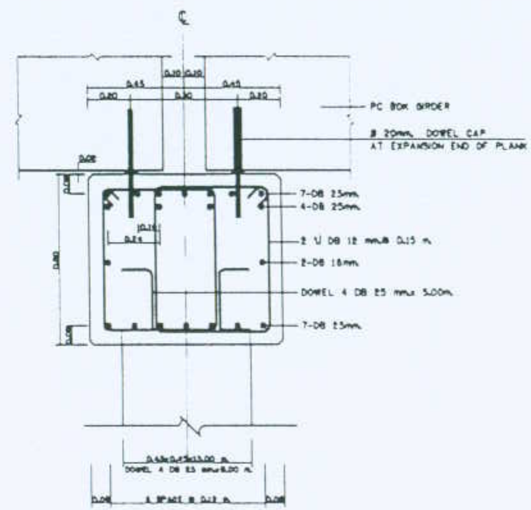


HALF SECTION ELEVATION



SECTION B-B

PILE/CAP CONNECTION DETAIL
SCALE 1 : 12.5



SECTION A-A

หมายเหตุ

เสาเข็ม คอร.บริเวณสะพาน ขนาด Ø0.45x0.45x15.00 ม.4DB25mmx6.00 ม.
สามารถรับน้ำหนักปลอดภัยได้ไม่น้อยกว่า 140 ตัน/ต้น
และสามารถรับโมเมนต์ดัดได้ไม่น้อยกว่า 10 ตัน-เมตร