Encl. 3/3 to 9A/TC/ITO/017
dd. 20-6-02 P-1 1 VOL 15 PAGES

ITO Joint Venture

Italian-Thai • Takenaka • Obayashi

CALCULATION SHEETS

FOR

TOWER CRANE FOUNDATION

DEVELOPED BY

ITALIAN-THAI DEVELOPMENT PUBLIC COMPANY LIMITED

TAKENAKA CORPORATION

OBAYASHI CORPORATION

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DESIGN CRITERIA

1. Structural Concrete f'_c = 240 ksc.
   f_c = 108 "

2. Reinforcing Steel
   - round bar grade SR.24 f_s = 1,200 ksc.
   - deformed bar grade SD.40 f_s = 1,600 "

3. Method of Analysis
   elastic stress design

4. Wind Load
   base on B.M.A. regulation

5. Tower Crane
   - MCA 501, 551 CT601, 602 (SRUCOMEDIL)
   - MC 120 (POTAIN)

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CRANE MODEL MC 501, 551 AND MC 120

summary of force refer to manufacturer's technical data (see attachment)

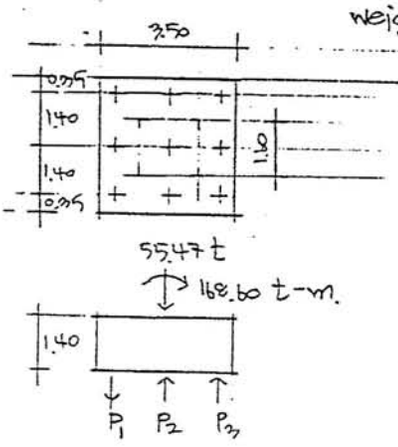
use maximum force of in service case and out of service case

crane model	V (t)	H (t)	M (t-m)
MCA 501, 551	55.47	9.90	168.60
Mc 120	55.47	8.98	141.76

Note maximum force from manufacturer's technical data are govern every load case during crane in service and out of service and both direction of jib, i.e. the jib is parallel to the side or parallel to the diagonal of the foundation pad.

LOADING ON PILE

1. The jib is parallel to the side of the foundation pad.



weight of foundation = $3.50 \times 3.50 \times 1.4 \times 2.4$

= 41.16 t

- reaction force on each pile

$P_1 = \frac{96.63}{9} - \frac{168.60 \times 1.40}{6 \times 1.40^2} = -9.33$ t

$P_2 =$ " $= 10.74$ t

$P_3 =$ " $+ \frac{168.60 \times 1.40}{6 \times 1.40^2} = 30.81$ t

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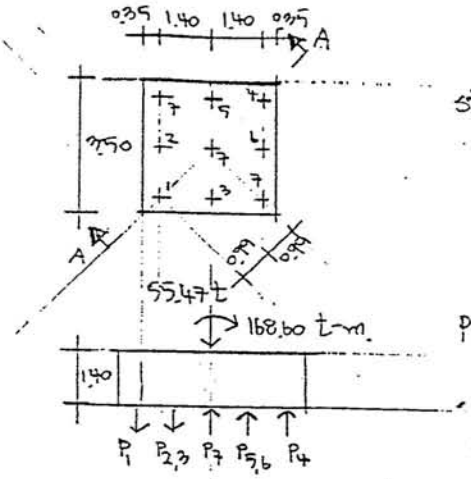
CRANE MODEL MC 501, 551 AND MC 120 (cont.)

2. The jib is parallel to the diagonal of the foundation pad.



weight of foundation = 44.16 t

steel pile 4-300×300×10×15 mm. $A = 119.80$ cm^2.

$I_A = 0.01198 \times 1.98^2 \times 2 + 0.01198 \times 0.99^2 \times 4 = 0.1409$ m^4

$P_1 = \frac{96.63}{9} - \frac{168.80 \times 1.98 \times 0.01198}{0.1409} = -17.65$ t

$P_{2,3} = \quad " \quad - \frac{168.80 \times 0.99 \times 0.01198}{0.1409} = -7.46$ t

$P_4 = \quad " \quad + \frac{168.80 \times 1.98 \times 0.01198}{0.1409} = 39.12$ t

$P_{5,6} = \quad " \quad + \frac{168.80 \times 0.99 \times 0.01198}{0.1409} = 24.93$ t

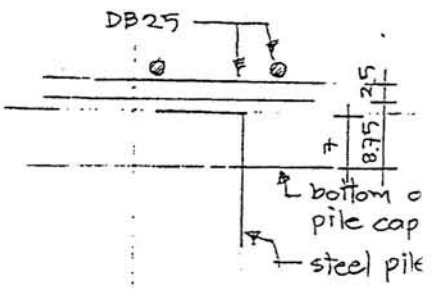
$P_7 = \quad " \quad = 10.74$ t

- check shear $\quad d = 140 - 87.5 - 2.5 = 128.75$ cm.

$V_c = 0.29\sqrt{240} \times 350 \times 128/1000$

$= 201.27$ t

$V = 3 \times 39.12 = 117.36$ t $< V_c$ ok.



- check bending moment

$M = 3 \times 39.12 \times 0.60 = 70.416$ t-m

$A_s = 70.416/1.60 \times 0.891 \times 1.28 = 38.59$ cm^2. 21 DB25 #

$A_{s\,min} = 0.002 \times 350 \times 140 = 98.00$ " ($A_s = 103.11$ cm^2.

$z_o = 3 \times 39.120/20.015 \times 0.891 \times 128 = 51.41$ cm. $\quad z_o = 165.06$ cm.)

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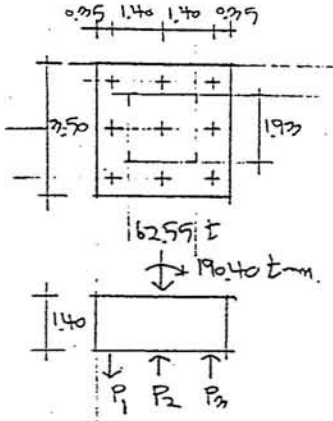
CRANE MODEL CT601, 602

maximum force refer to manufacturer's technical data

$V = 62.55$ t , $M = 190.40$ t-m.

LOADING ON PILE

1. The jib is parallel to the side of the foundation pad.



weight of foundation = 41.16 t

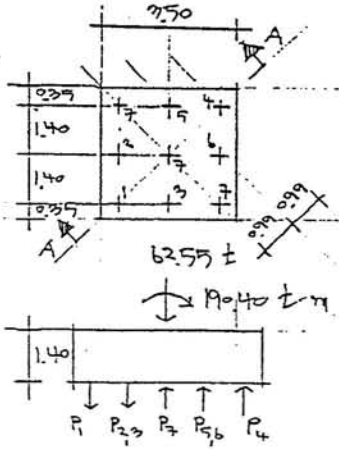
reaction force on each pile

$P_1 = \frac{103.71}{9} - \frac{190.40 \times 1.40}{6 \times 1.40^2} = -11.15$ t

$P_2 =$ " $= 11.52$ t

$P_3 =$ " $+ \frac{190.40 \times 1.40}{6 \times 1.40^2} = 34.19$ t

2. The jib is parallel to the diagonal of the foundation pad.



steel pile H-300×300×10×15 mm. $A = 119.80$ cm^2

$I_A = 0.1409$ m^4

$P_1 = \frac{103.71}{9} - \frac{190.40 \times 1.98 \times 0.01198}{0.1409} = -20.53$ t

$P_4 =$ " $+$ " $= 43.57$ t

$P_{2,3} =$ " $- \frac{190.40 \times 0.99 \times 0.01198}{0.1409} = -4.51$ t

$P_{5,6} =$ " $+$ " $= 29.55$ t

$P_7 =$ " $= 11.52$ t

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CRANE MODEL CT 601, 602 (cont.)

- check shear

$d = 140 - 8.75 - 2.5 = 128.75$ cm.

$V_c = 0.29\sqrt{240} \times 350 \times 128/1000 = 201.27$ t

$V = 3 \times 43.57 = 130.71$ t $< V_c$ ok.

- check bending moment

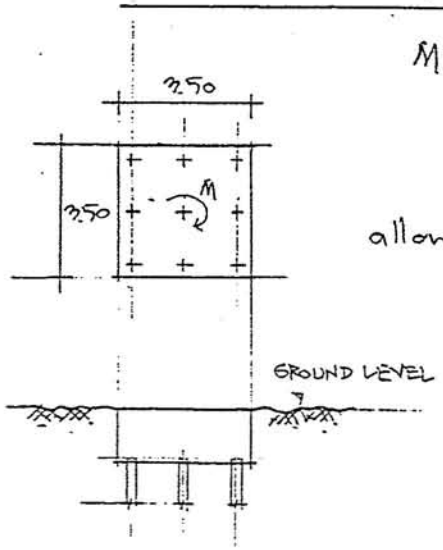
$M = 3 \times 43.57 \times 0.45 = 58.82$ t-m.

$A_s = 58.82/1.60 \times 0.891 \times 1.28 = 32.23$ cm.^{2} 21 DB25

$A_{s\,min.} = 0.002 \times 350 \times 140 = 98$ " ($A_s = 107.11$ cm.^{2}

$Z_o = 3 \times 43570/2.015 \times 0.891 \times 128 = 57.26$ cm. $Z_o = 165.06$ cm.)

RECHECK PILE CAP DUE TO TORSIONAL MOMENT



$M = 14.68$ t-m.

$v_t = \frac{3.5 \times 14.68 \times 100{,}000}{350^2 \times 350} = 0.12$ ksc.

allowable $v_t = 1.32\sqrt{240} = 20.45 > 0.12$ " ok.

∴ Not require stirrup.

$A_s = \frac{M_t \cdot Z}{2A_o \cdot f_s}$ $\quad Z = \frac{335+335}{2} = 335$ cm.

$= \frac{1{,}468{,}000 \times 335}{2 \times 112{,}225 \times 1{,}600} = 1.37$ cm.^{2}

provide 1 DB25

($A_s = 4.91$ cm.^{2})

Note By inspection torsional moment does not make steel pile critic

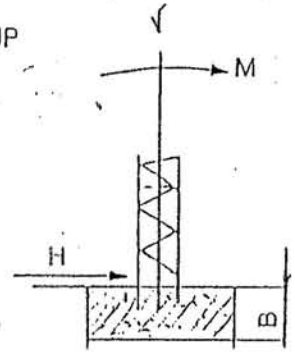
JIB LENGTH 55 m

HEIGHT UNDER HOOK m	TELESCOPING CAGE POSITION	NUMBER AND TYPE OF TOWER SECTIONS A 150 x 16	TR2		WIND IN SERVICE V (t)	H (t)	M (tm)	WIND OUT OF SERVICE V (t)	H (t)	M (tm)	PRESSURE MAX KG/CM2 S	FS	PLINTH SIZE (m) A x B	*
18.25	UP	3	1		50.38	1.8	101.8	45.74	7.06	56.59	1.38		4.6 x 1.3	4
22.00	UP	4	1		51.65	1.96	108.9	46.94	7.76	85.52	1.44		4.6 x 1.3	
25.75	UP	5	1		52.92	2.12	116.5	48.14	8.48	117.7		1.28	5.0 x 1.3	4.5
29.50	UP	6	1		54.20	2.28	124.8	49.34	9.24	153.8		1.48	5.0 x 1.3	
33.25	UP	7	1		55.47	2.44	133.7	50.54	10	193.6		1.44	5.4 x 1.3	5
→ 33.25	DOWN	7	1		55.47	2.44	133.7	50.54	9.90	168.6		1.31	5.3 x 1.3	
37.00	DOWN	8	1		56.74	2.61	143.1	51.74	10.7	199.1		1.47	5.4 x 1.3	

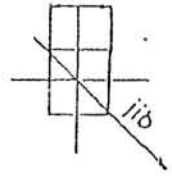
SF = WITHOUT TELESCOPING CAGE
V = VERTICAL LOAD
H = HORIZONTAL THRUST
M = TOTAL MOMENT
S = IN SERVICE
FS = OUT OF SERVICE

* MIN. PLINTH SIDE "A" PRESSURE TO BE VERIFIED

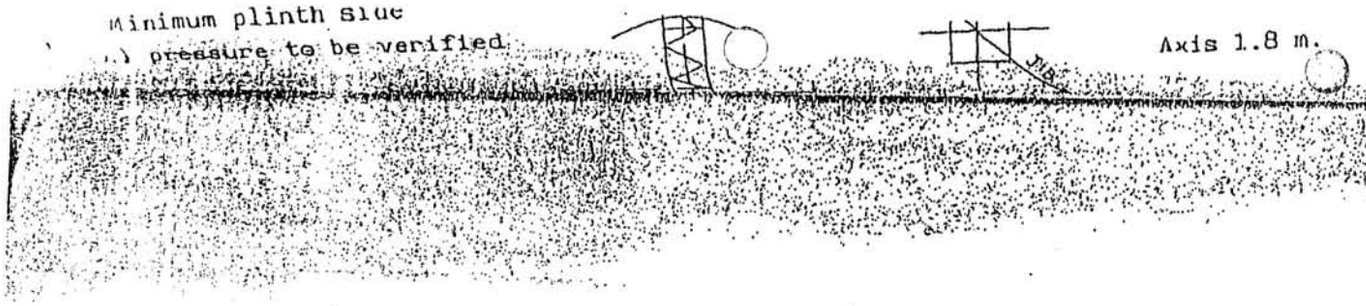
CRANE IN SERVICE: FORCES AT GROUND CALCULATED WITH TELESCOPING CAGE UP



1.6 x 1.6 external side



ZONE III

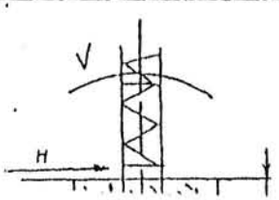


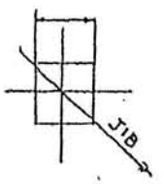
WIND III ZONE

CRANE TYPE CT 602 — STATIC WITHOUT CHASSIS - "R" VERSION

HEIGHT UNDER HOOK (m)	TELESCOP. CAGE POSITION	TOWER ELEMENTS QUANTITY AND TYPE 120x11	120x11		WIND 'IN SERVICE' V (t)	H (t)	M (tm)	WIND 'OUT OF SERVICE' V (t)	H (t)	M (tm)	PRESSURE Max./Kg./cm2 S	FS	PLINTH DIMENSIONS	*
22.15	HIGH	5			58.05	2.09	161.6	56.49	8.34	74.38	1.52		5 x 5 x 1.3	
25.90	HIGH	6			59.55	2.24	169.8	57.91	9.01	109.3	1.57		5 x 5 x 1.3	4.6
29.65	HIGH	7			61.05	2.39	178.5	59.32	9.73	148.0	1.62		5 x 5 x 1.3	
33.40	HIGH	8			62.55	2.53	187.8	60.74	10.4	190.4		1.42	5.5 x 5.5 x 1.3	
37.15	HIGH	8	1		64.05	2.68	197.6	62.15	11.2	236.5		1.60	5.5 x 5.5 x 1.3	5
40.90	LOW	8	2		65.70	2.83	208.0	63.71	11.8	234.5		1.60	5.5 x 5.5 x 1.3	
44.65	LOW	8	3		67.35	2.98	219.0	65.27	12.6	280.5		1.49	6 x 6 x 1.3	5.5
48.40	LOW	8	4		69.01	3.13	230.5	66.83	13.3	330.1		1.64	6 x 6 x 1.3	

* Minimum plinth side
(A) pressure to be verified





External side 1.93 m.
Axis 1.8 m.

2. 3. 2. CALCULATION OF THE CONCRETE BLOCKS

2. 3. 2. 1. IN SERVICE

H.S.C	WEIGHT	MAXIMUM MOMENT	SHEARING FORCE	MAX. REACTIONS TENSION	COMPRESSION
8.00	46195	66748	1505	19958	43056
11.00	47345	70520	1638	21451	45124
14.00	48495	74691	1771	23133	47380
17.00	49645	79261	1903	25002	49825
20.00	50615	84024	2025	27008	52316
23.00	51585	89907	2147	29543	55335
26.00	52555	97728	2268	32992	59269
29.00	53525	106167	2390	36733	63496
32.00	54495	115186	2511	40748	67995
→ 35.00	55465	124749	2633	45019	72752
38.00	56435	134829	2754	49535	77752
41.00	57405	145401	2876	54283	82985
44.00	58375	156446	2997	59254	88441
47.00	59345	167949	3119	64441	94114

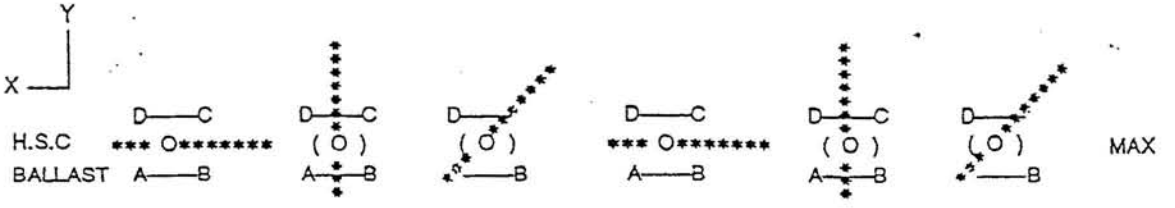
2. 3. 2. 2. OUT OF SERVICE

H.S.C	WEIGHT	MAXIMUM MOMENT	SHEARING FORCE	MAX. REACTIONS TENSION	COMPRESSION
8.00	40195	59226	4050	17907	38005
11.00	41345	59226	4475	17620	38292
14.00	42495	59226	4900	17332	38580
17.00	43645	59226	5325	17045	38867
20.00	44615	59226	5714	16802	39110
23.00	45585	59226	6661	16560	39352
26.00	46555	69126	7373	20991	44268
29.00	47525	91734	7908	31420	55182
32.00	48495	115946	8443	42606	66854
→ 35.00	49465	141762	8978	54550	79283
38.00	50435	169183	9512	67251	92469
41.00	51405	198208	10047	80710	106412
44.00	52375	228838	10582	94925	121113
47.00	53345	261072	11117	109898	136571

2. 3. REACTIONS ON THE FIXING ANGLES MC120 NF P16A CAN BE TELESCOPED

2. 3. 1. REACTIONS ON THE FIXING ANGLES

2. 3. 1. 1. REACTIONS IN SERVICE

REAR WIND / JIB						PERPENDICULAR						
												
H.S.C	D—C	D—C	D—C			D—C	D—C	D—C				MAX
BALLAST	*** O*******	(O)	(O)			*** O*******	(O)	(O)				
	A—B	A—B	B			A—B	A—B	B				
M	D	C										
T	A	B										
	TX	TY	TORSIONAL MOMENT 14677 m.daN TX TY : TOTAL SHEARING FORCE (torsion not inclued)									
8.0	11	-34	-34	-34	-12	-43	-2	-39	-21	-39	-12	-41	-43
	10	-34	11	10	20	-12	16	-21	16	-2	18	-12	
	1.4	0.0	0.0	1.4	1.1	1.0	0.0	2.9	2.9	0.0	0.9	2.8	
11.0	12	-35	-35	-35	-12	-45	-5	-42	-19	-42	-12	-43	-45
	11	-35	12	11	21	-12	18	-19	18	-5	19	-12	
	1.5	0.0	0.0	1.5	1.2	1.1	0.0	3.0	3.0	0.0	0.6	3.0	
14.0	13	-36	-36	-37	-12	-47	-8	-45	-16	-45	-12	-45	-47
	12	-37	13	12	23	-12	21	-16	21	-8	21	-12	
	1.7	0.0	0.0	1.7	1.3	1.2	0.0	3.1	3.1	0.0	0.4	3.2	
17.0	14	-38	-38	-39	-12	-50	-11	-48	-14	-48	-12	-48	-50
	13	-39	14	13	25	-12	24	-14	24	-11	24	-12	
	1.8	0.0	0.0	1.8	1.3	1.3	0.0	3.2	3.2	0.0	0.1	3.3	
20.0	15	-40	-40	-40	-13	-52	-14	-51	-11	-51	-13	-52	-52
	15	-40	15	15	27	-13	26	-11	26	-14	27	-13	
	1.9	0.0	0.0	1.9	1.4	1.4	0.0	3.3	3.3	0.0	0.2	3.4	
23.0	17	-42	-42	-42	-13	-55	-17	-55	-8	-55	-13	-55	-55
	16	-42	17	16	29	-13	29	-8	29	-17	30	-13	
	2.0	0.0	0.0	2.0	1.5	1.5	0.0	3.4	3.4	0.0	3.5	0.4	
26.0	18	-44	-44	-44	-13	-57	-21	-58	-5	-58	-13	-59	-59
	17	-44	18	17	31	-13	32	-5	32	-21	33	-13	
	2.1	0.0	0.0	2.1	1.6	1.6	0.0	3.5	3.5	0.0	0.7	3.6	

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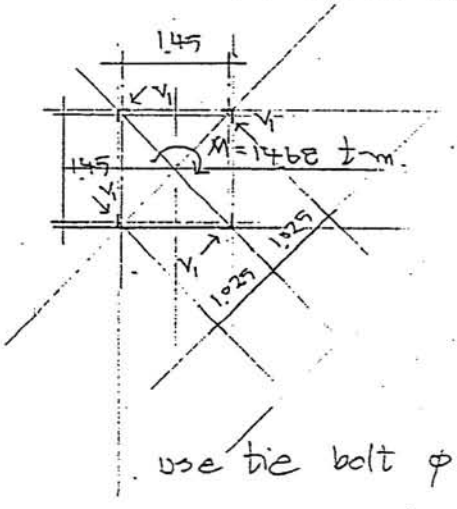
BASE FIXING

Summary of force refer to manufacturer's technical data (see attachment) use maximum force of in service case and out of service case

crane model	H (t)	M (t-m)	mast size (m.)	tension on each bolt (t)
MCA 501, 551	9.90	168.60	1.50	14.05
MC 120	8.98	141.76	1.45	12.22
CT601, 602	10.40	190.40	1.80	13.22

torsional moment = 14.677 t-m. (in service case) refer to manufacturer's tech data (see attachme

Note By inspection torsional moment for out of service is not critical.



$V_1 = 14.68/1.025 \times 4 = 3.58$ t/leg

$V = 3.58/4 = 0.895$ t/bolt

due to horizontal force

$V = 10.40/4 \times 4 = 0.65$ t/bolt

$\Sigma V = 0.895 + 0.65 = 1.545$

use tie bolt ϕ 45 mm. $A = 12.57$ cm^2. $F_y = 2{,}400$ ksc.

allowable tension $= 0.6 \times 2400 \times 12.57/1000 = 18.10$ t > 14.05

allowable shear $= 0.4 \times 2400 \times 12.57/1000 = 12.07$ t > 1.545

combine force

$$\frac{14.05}{18.10} + \frac{1.545}{12.07} = 0.904 < 1.00 \quad \text{ok}$$

KING POST CAPACITY

H - 300x300 mm. x 19.00 m.

- END BEARING

$Q_b = 9 \cdot C_b \cdot A_b$

$N = 28 \rightarrow C_b = 18.50$ t/m². (ref. N/Q_b chart, use Terzaghi & Peck's line)

$Q_b = 9 \times 18.5 \times 0.30 \times 0.30$

$= 14.985$ t

- SKIN FRICTION

$Q_s = \Sigma K \cdot C_b \cdot P \cdot \Delta L$

$= 0.96 \times 1.75 \times 4 \times 0.30 \times 3.00 + 4 \times 0.30 \times 1.5\,(0.97 \times 0.85 + 0.97 \times 0.95 + 0.96 \times 1.$
$+ 0.95 \times 1.90 + 0.94 \times 2.35 + 0.91 \times 3.15 + 0.81 \times 5.35) + 0.42 \times 13.5 \times 1.20 \times 3.5$

↑ N = 20

$= 5.48 + 25.661 + 23.814.$

$= 55.523$ t

$\Sigma Q = Q_b + Q_s$

$= 14.985 + 55.523 = 70.508$ t

For temporary work purpose F.S. = 1.5

Allowable capacity of each post = 70.508/1.5

= 47.01 t

STS ENGINEERING CONSULTANTS CO.,LTD.
SUMMARY OF TEST RESULTS

PROJECT SBIA AIR PASSENGER TERMINAL & FACILITIES | LOCATION NONGNGUEHAO, SAMUTPRAKARN

DATE 20/01/97 | BORING No. BH-6 | JOB No. 6025 | BY PT' | OBSERVED W.L. -2.00 M

SAMPLE No.	DEPTH M. FROM	DEPTH M. TO	WATER CONTENT %	ATTERBERG LIMIT % LL.	ATTERBERG LIMIT % PL.	ATTERBERG LIMIT % PI.	WET UNIT WEIGHT γ_B	SIEVE ANALYSIS % FINER No. 3/8"	No. 4	No. 10	No. 40	No. 200	CLASSIFICATION	UNDRAINED SHEAR STRENGTH t/m² UNCONFINED SHEAR Qu/2	Qu'/2	TORE VANE SHEAR Qv	Qv'	UU TEST Su	POCKET PENETROMETER 1/2Qp	STANDARD PENETRATION (N) (blow/ft)	Direct Shear Test $\bar{C}$	Direct Shear Test $\bar{\emptyset}°$
ST-01	1.50	2.00	43.50				1.70		1.78				CH	2.70	2.70	7.50	0.40		3.8			
ST-02	3.00	3.50	93.50				1.46						CH			1.30	0.40		1.3			
ST-03	4.50	5.00	118.80	92.20	32.50	59.70	1.41		1.44				CH	0.80	0.85	1.10	0.20		1.3			
ST-04	6.00	6.50	98.40				1.47						CH	0.90		1.50	0.40		1.3			
ST-05	7.50	8.00	117.40				1.41		1.41				CH	0.80	0.95	1.90	0.80		1.3			
ST-06	9.00	9.50	116.30				1.41						CH	1.10		1.80	0.50		1.3			
ST-07	10.50	11.00	88.00				1.50						CH	1.60	1.90	2.30	0.60		1.3			
ST-08	12.00	12.50	94.50				1.48		1.49				CH	2.20		3.00	1.00		1.3			
ST-09	13.50	14.00	66.80				1.62						CH	2.50	3.15	2.80	0.90		1.3			
ST-10	15.00	15.50	59.00				1.67		1.65				CH	3.80		4.00	1.70		3.8			
ST-11	16.50	17.00	35.50				1.86		1.86				CH	6.90	6.90	7.40	2.10		7.5			
SS-12	18.00	18.45	38.80	82.80	33.30	49.50	1.79		1.79				CH		8.0				12.5	13		
SS-13	20.00	20.45	28.80										CH						15.0	28		
SS-14	22.00	22.45	25.80						100	95	92	62	CL							10		
SS-15	24.00	24.45	23.40							100	99	19	SM							30	0	32
SS-16	26.00	26.45	23.50										SM							25		
SS-17	28.00	28.45	23.80								100	12	SM-SP							37		
SS-18	30.00	30.28	18.10										SM-SP							50/5"		
SS-19	32.00	32.45	12.40						100	84	41	12	SM-SP							55	0	33
SS-20	34.00	34.45	21.20										SM-SP							29		

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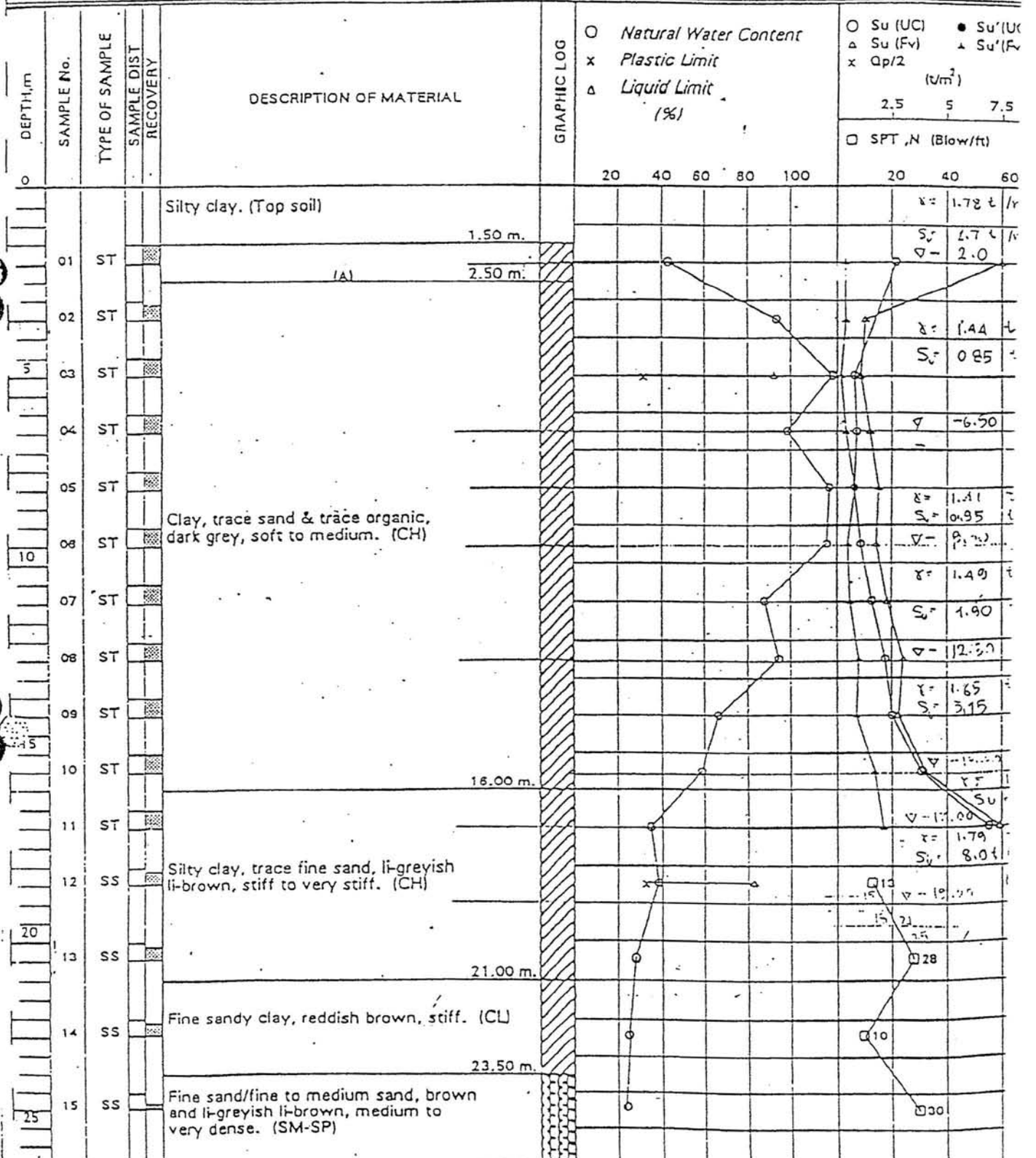
LOG OF BORING No. BH-6

PROJECT : SBIA AIR PASSENGER TERMINAL & FACILITIES

LOCATION : NONGNGUEHAO, SAMUTPRAKARN

CLIENT :



STS ENGINEERING CONSULTANTS CO.,LTD.

BORING STARTED. 23/11/96 | RIG. PORT | WL -2.00 M. AFTER 8(

BORING FINISHED. 24/11/96 | FOREMAN SN. | JOB No. 6025

Title	Date	Dept.	Page
SUVARNABHUMI AIRPORT	03/02		
TOWER CRANE FOUNDATION	Signature		

SUMMARY OF FORCE Note use max. force of all case.

CRANE	V(t)	H(t)	M(t-m)	crane size(m.)	tension on each bolt t
MCA 551, 501	55.47	9.90	186.60	1.50	14.05
CT 602, 601	52.69	10.30	190.20	1.20	6.604
MC 120	55.47	8.98	141.76	1.55	11.43

fabrication weight = $3.50 \times 3.50 \times 1.4 \times 2.5 = 41.16$ t

total = $52.69 + 41.16 = 103.85$ t

Reaction force on each pile

$P_1 = -\frac{103.85}{9} - \frac{190.20 \times 1.4}{6 \times 1.4^2} = -11.10$ t

$P_2 = \frac{}{4} + \frac{}{4} = 34.18$ t ← max.

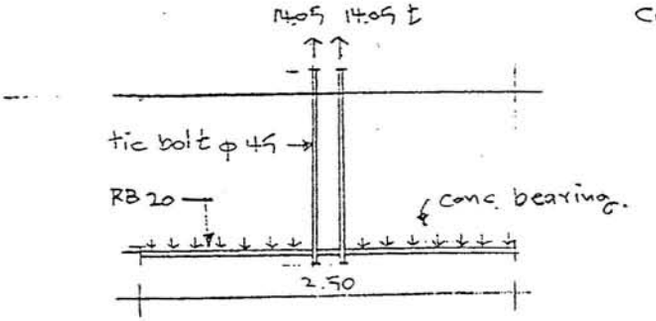
$P_3 = \frac{}{4} = 11.54$ t

By inspection max. force on each pile less than previous cal. ok.

CONNECTION tension = 14.05 t/bolt

use tie bolt ϕ 45 mm. A = 1590 cm^2 F_y = 2400 ksc.

allowable tension = $0.6 \times 2400 \times 1590/1000 = 22.896$ t ok.



conc. bearing capacity

$= 0.25 \times 240 \times 2 \times 250/1000$

$= 30$ t > 28.10 t ok.