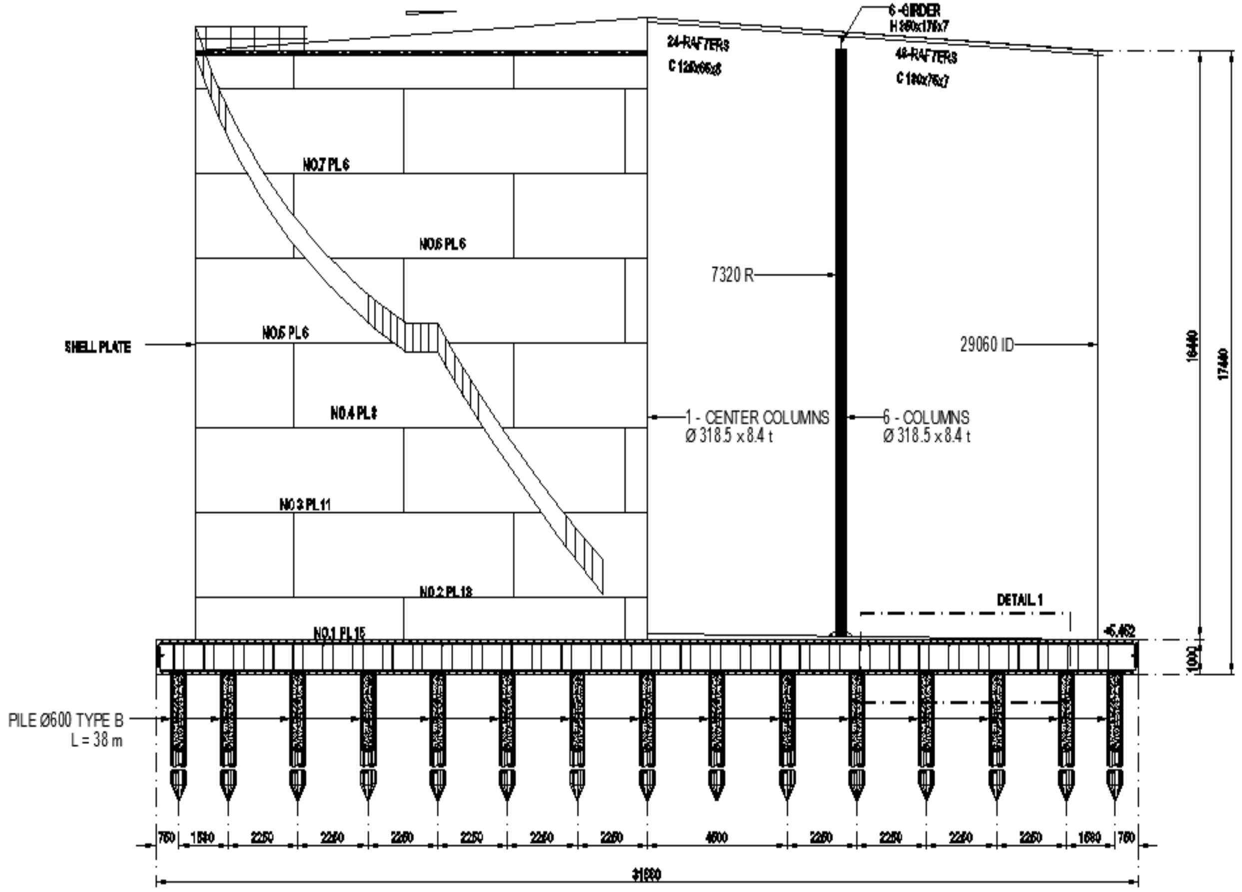
Rekayasa Pondasi II

Dosen : Sherly Meiwa ST., MT.

Jurusan Teknik Sipil
Universitas Komputer Indonesia
Bandung, 2020

Contoh Proyek

DESAIN STRUKTUR
DAN PONDASI TANKI 610-TK-213 AREA OM PT. PERTAMINAPERSERO RU II DUMAI



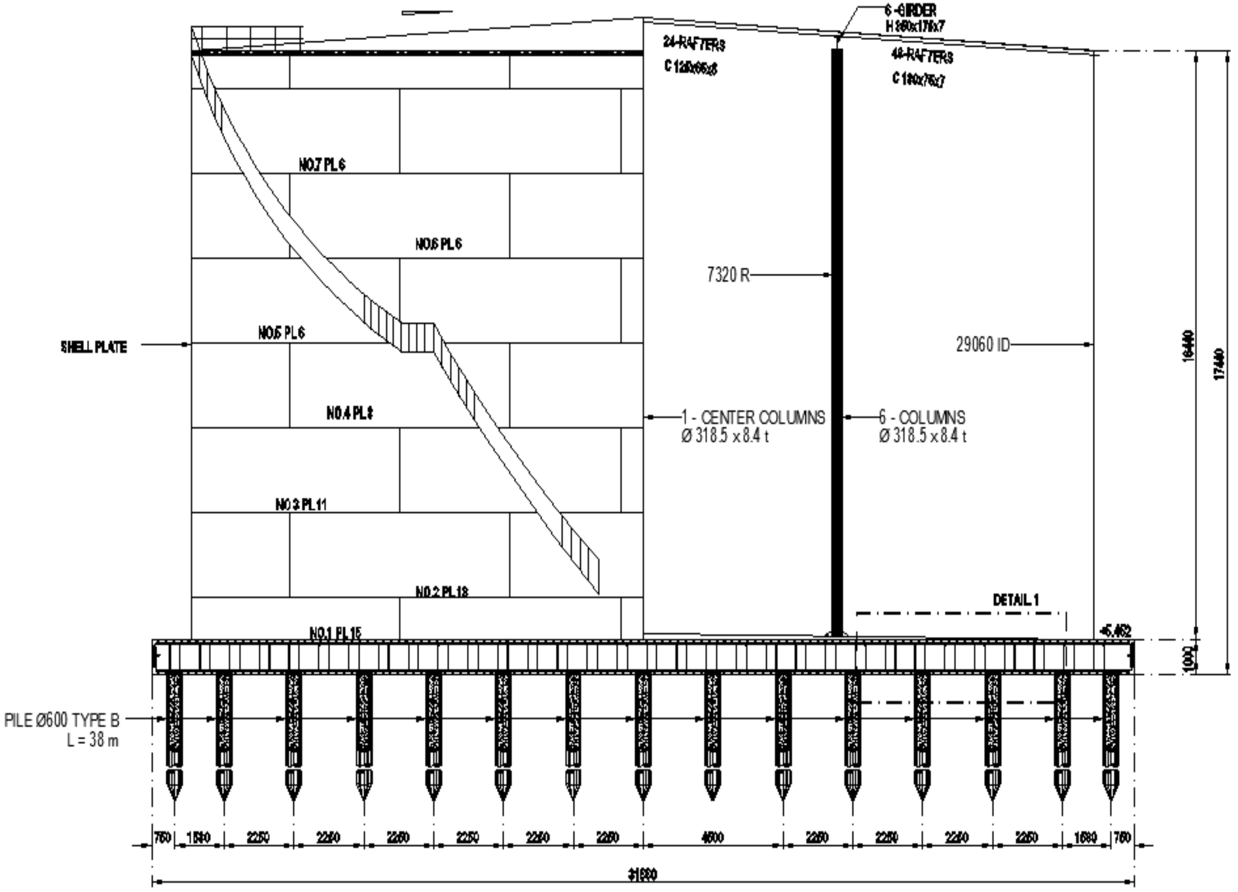
Tank Diameter	:	48310 mm
Tank Height	:	12240 mm
Pile Cap/Slab Diameter	:	48310 mm
Tank Steel Thickness	:	20 mm
Capacity	:	10000 kL
Unit weight of Solar	:	8.7 kN/m^3
Unit weight of water	:	10 kN/m^3

Outline

Session 1	1. Contoh Proyek 2. Langkah Membuat Soil Stratigrafi Tanah 3. Definisi Pondasi Dalam (Fungsi dan Jenisnya) 4. Rumus Dasar menghitung daya dukung pondasi tiang
Session 2-3	5. Daya dukung pondasi tiang pancang pada tanah pasir 6. Daya dukung pondasi tiang pancang pada tanah lempung 7. Daya dukung pondasi tiang bor pada tanah pasir 8. Daya dukung pondasi tiang bor pada tanah lempung
Session 4	9. Tugas I axial single Pile 10. Negative skin friction
Session 5	11. Group pile 12. Tugas 2 Group Pile

Contoh Proyek

DESAIN STRUKTUR
DAN PONDASI TANKI 610-TK-213 AREA OM PT. PERTAMINAPERSERO RU II DUMAI



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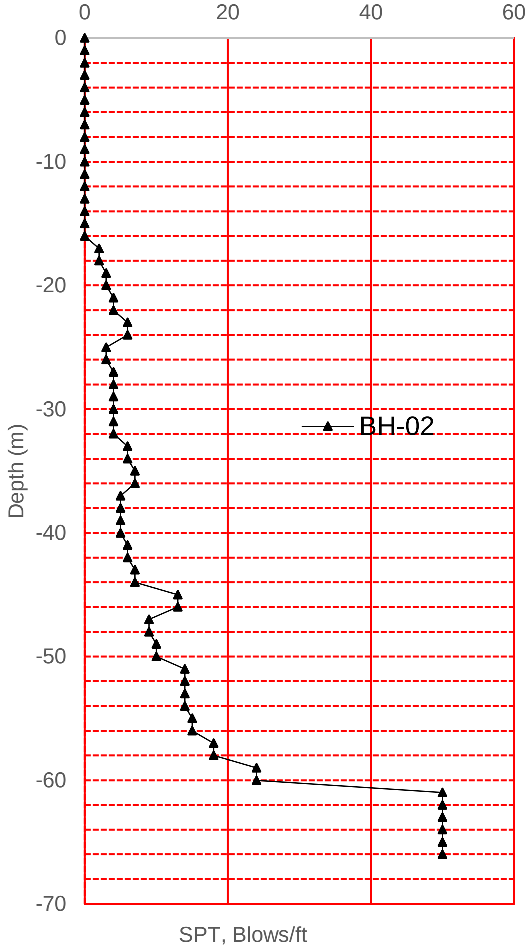
Contoh Proyek

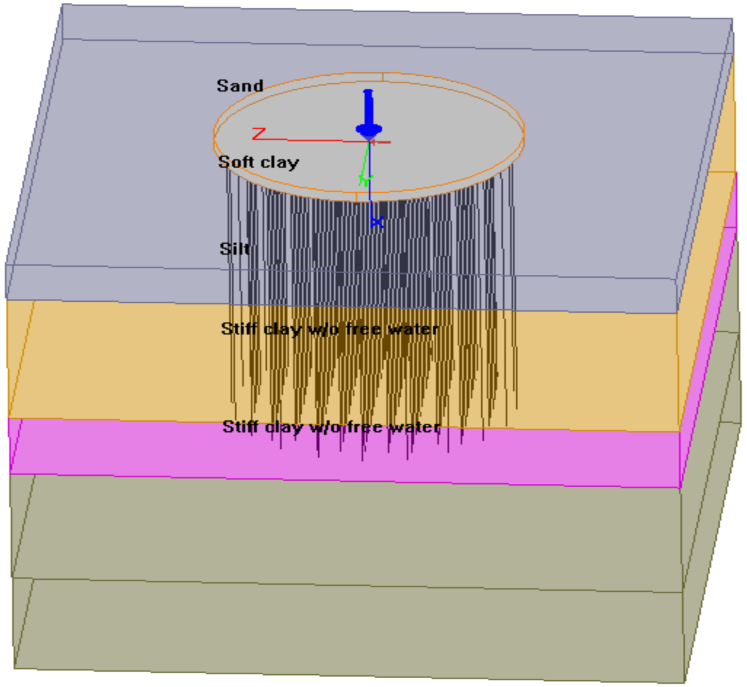
Parameter Tanah Desain

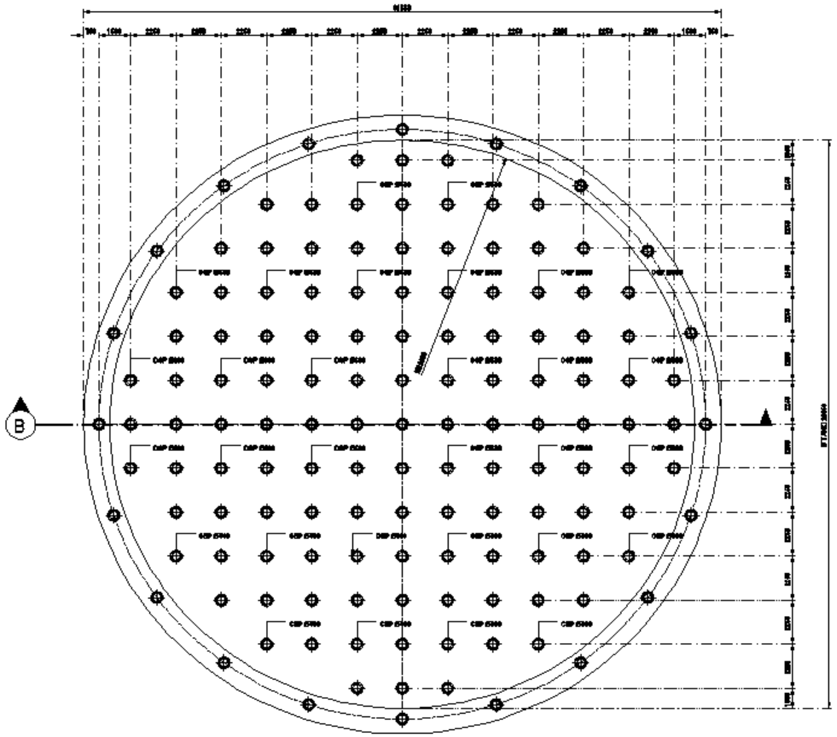


Tabel 3.5 Parameter Tanah Desain

No.	Jenis Tanah	Kedalaman (m)	NSPT (avg)	g , (kN/m³)	Kohesif (cu) (kN/m2)	Friction Angle (f)	E50 (kN/m2)
1	Organic Clay	00.00 – 03.00	1	4.8	6	0	0.02
2	Very Soft Clay	03.00 – 17.00	1	7	6	0	0.02
3	Soft Clay	17.00 – 32.00	4	9.2	22	0	0.02
4	Soft to Medium Clay	32.00 – 52.00	8	8.2	49	0	0.01
5	Stiff Clay	52.00 - 60.00	24	9	96	0	0.005
6	Hard Clay	60.00-66.00	50	19	144	0	0.004

Contoh Proyek



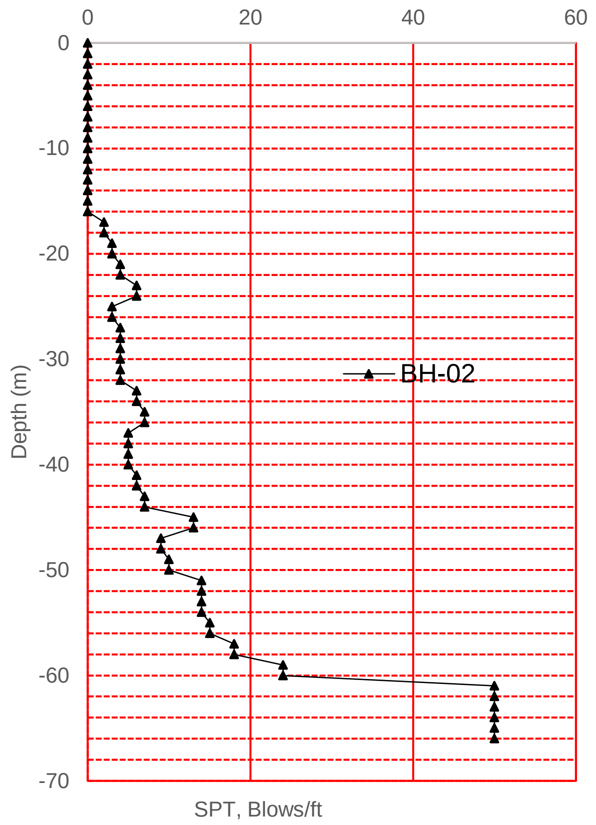


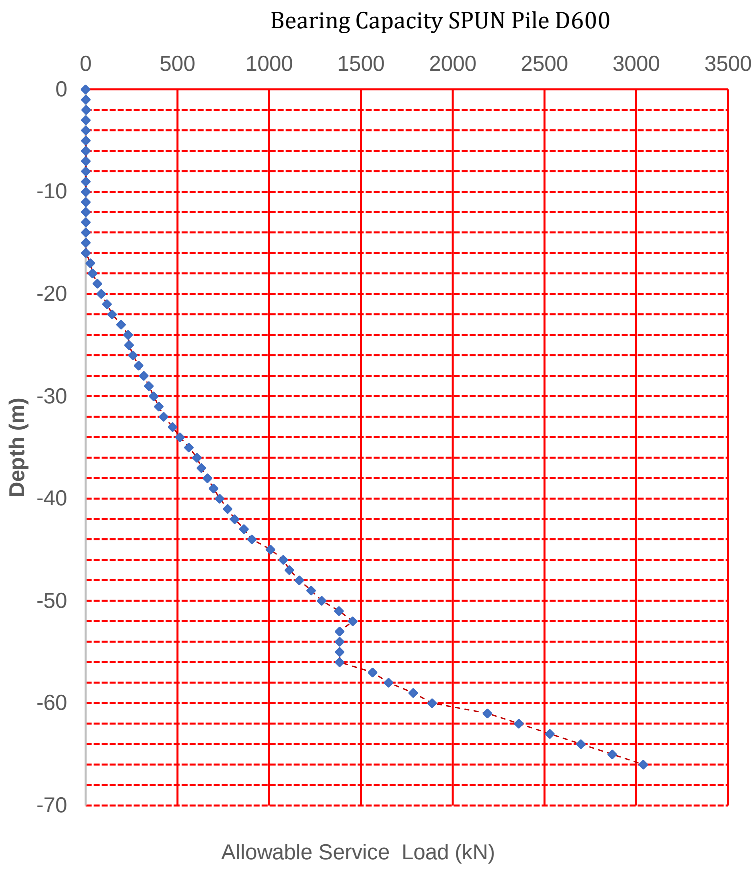
Type of Foundation	Load [kN]		Allowable Single Pile Capacity [kN]		Efficiency	Number Of Pile AFTER Design	(SERVICE LOAD)		(ULTIMATE LOAD)	
	Service Load (Operating)	Ultimate Load	Service Load (Operating) sf=2.5	Ultimate Load sf=1.67			Group pile capacity	Status	Group pile capacity	Status
Spun Pile	219295	333000	1259.295	1885.172	0.79	232	229100	OK	345514	OK
Stell Pipe Pile	219295	333000	1517.6	2271.9	0.75	201	226125	OK	342482	OK

Contoh Proyek

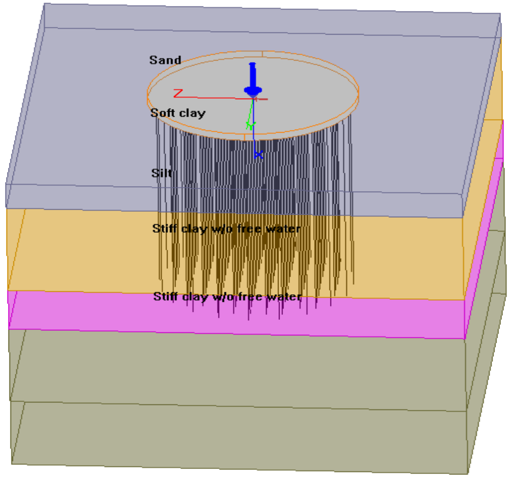
Depth (m)	Soil Type	N-SPT (blows/30cm)	γ (kN/m³)	c (kN/m²)	ϕ (degree)	Layer Thickness	σ_d (kN/m²)	σ_d ave (kN/m²)	Adhesion Factor (α_p)	qs (kN/m²)	Outer Per (m²)	Qs-outer (kN)	Nc	Nq	qb (kN/m²)	Qb (kN)	Q ultimate (kN)		Q ULS (kN)		Q SLS (kN)	
																			Tekan	Tarik	Tekan	Tarik
0						0 m	0	0	1.00	0	0.00	0	5.1	1.0	0	0	0		0	0		
1	Very Soft Peat	0	12	1	0	1 m	2	1	1.00	1	1.88	2	5.1	1.0	9	1	3		2	5		
2	Very Soft Peat	0	12	1	0	1 m	4	3	1.00	1	1.88	4	5.1	1.0	9	1	5		3	9	2	1
3	Very Soft Clay	0	12	0	0	1 m	6	5	1.00	0	1.88	4	9.0	1.0	0	0	4		2	13	2	5
4	Very Soft Clay	0	12	0	0	1 m	8	7	1.00	0	1.88	4	9.0	1.0	0	0	4		2	17	2	9
5	Very Soft Clay	0	12	0	0	1 m	11	9	1.00	0	1.88	4	9.0	1.0	0	0	4		2	20	2	12
6	Very Soft Clay	0	12	0	0	1 m	13	12	1.00	0	1.88	4	9.0	1.0	0	0	4		2	24	2	16
7	Very Soft Clay	0	12	0	0	1 m	15	14	1.00	0	1.88	4	9.0	1.0	0	0	4		2	28	2	20
8	Very Soft Clay	0	12	0	0	1 m	17	16	1.00	0	1.88	4	9.0	1.0	0	0	4		2	32	2	24
9	Very Soft Clay	0	12	0	0	1 m	19	18	1.00	0	1.88	4	9.0	1.0	0	0	4		2	36	2	27
10	Very Soft Clay	0	12	0	0	1 m	21	20	1.00	0	1.88	4	9.0	1.0	0	0	4		2	39	2	31
11	Very Soft Clay	0	12	0	0	1 m	23	22	1.00	0	1.88	4	9.0	1.0	0	0	4		2	43	2	35
12	Very Soft Clay	0	12	0	0	1 m	25	24	1.00	0	1.88	4	9.0	1.0	0	0	4		2	47	2	39
13	Very Soft Clay	0	12	0	0	1 m	27	26	1.00	0	1.88	4	9.0	1.0	0	0	4		2	51	2	43
14	Very Soft Clay	0	12	0	0	1 m	29	28	1.00	0	1.88	4	9.0	1.0	0	0	4		2	54	2	46
15	Very Soft Clay	0	12	0	0	1 m	32	30	1.00	0	1.88	4	9.0	1.0	0	0	4		2	58	2	50
16	Very Soft Clay	0	12	0	0	1 m	34	33	1.00	0	1.88	4	9.0	1.0	0	0	4		2	62	2	54
17	Very Soft Clay	2	13	12	0	1 m	37	35	1.00	12	1.88	26	9.0	1.0	108	17	43		26	75	17	64
18	Very Soft Clay	2	13	12	0	1 m	40	38	1.00	12	1.88	49	9.0	1.0	108	17	66		40	88	26	74
19	Very Soft Clay	3	13	18	0	1 m	43	41	1.00	18	1.88	83	9.0	1.0	162	25	108		65	106	43	87
20	Very Soft Clay	3	13	18	0	1 m	47	45	1.00	18	1.88	117	9.0	1.0	162	25	142		85	124	57	101
21	Soft Clay	4	14	24	0	1 m	50	48	1.00	24	1.88	162	9.0	1.0	216	34	196		118	147	78	117
22	Soft Clay	4	14	24	0	1 m	54	52	1.00	24	1.88	207	9.0	1.0	216	34	241		145	170	97	133
23	Medium Clay	6	16	36	0	1 m	60	57	0.96	34	1.88	272	9.0	1.0	324	51	323		194	201	129	155
24	Medium Clay	6	16	36	0	1 m	66	63	0.96	34	1.88	337	9.0	1.0	324	51	388		233	232	155	177
25	Very Soft Clay	3	13	18	0	1 m	70	68	1.00	18	1.88	371	9.0	1.0	162	25	396		238	250	159	191
26	Very Soft Clay	3	13	18	0	1 m	73	71	1.00	18	1.88	405	9.0	1.0	162	25	430		258	268	172	204
27	Soft Clay	4	14	24	0	1 m	77	75	1.00	24	1.88	450	9.0	1.0	216	34	484		290	291	194	220

Contoh Proyek





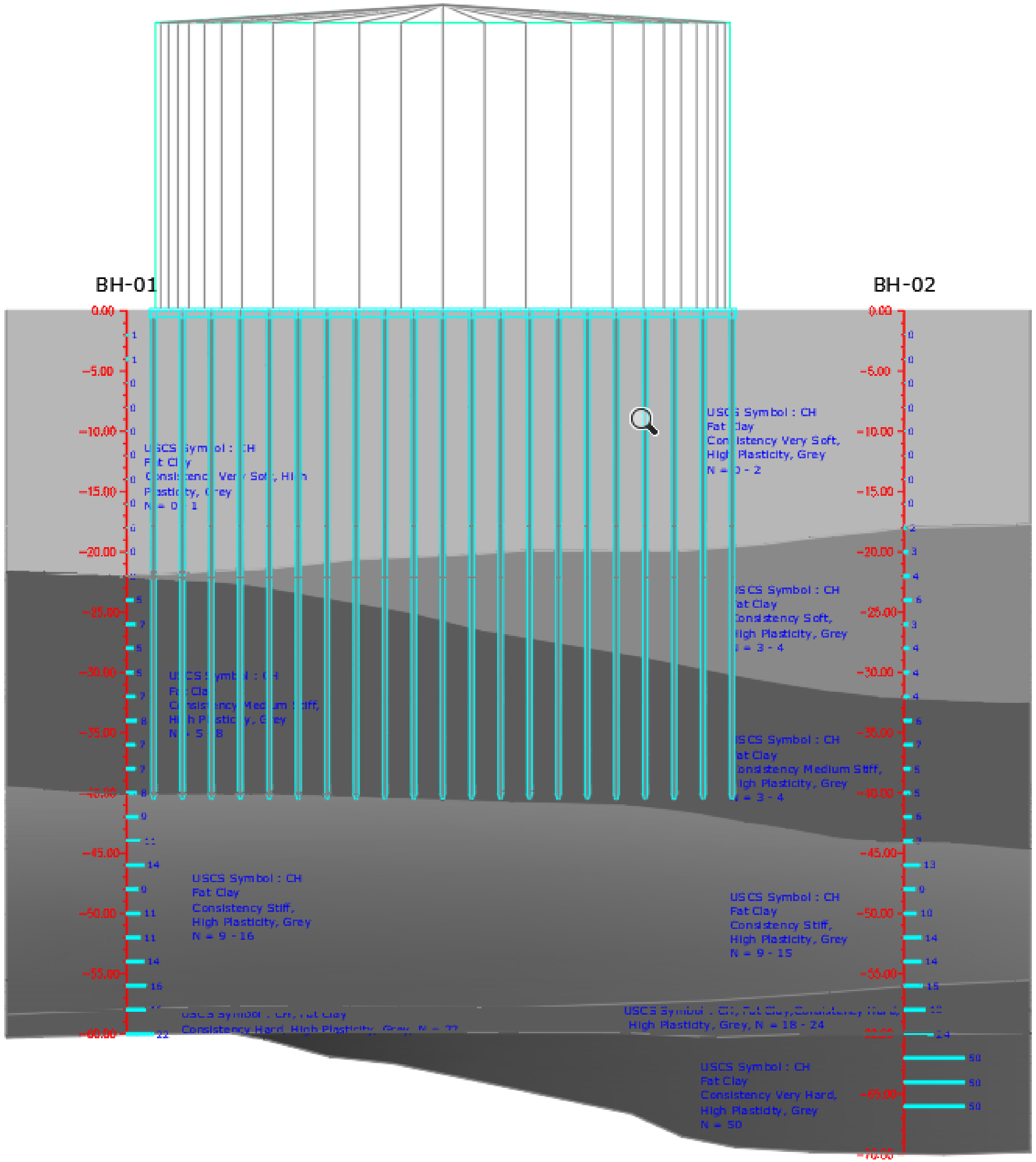
Contoh Proyek



Diameter tiang = 600 mm
Jumlah Tiang = 224 tiang
Kedalaman Tiang = 60 m

Type of Foundation	Diameter [mm]	Thickness [mm]	Embedded Length [m]	Ultimate Axial Capacity to Single Pile [kN]	Load [kN]			Allowable Single Pile Capacity [kN]			Number Of Pile before efficiency		
					Service Load (Hydrostest)	Service Load (Operating)	Ultimate Load	Service Load (Hydrostatic) SF=2	Service Load (Operating) sf=2.5	Ultimate Load sf=1.67	Service Load (Hydrostest)	Service Load (Operating)	Ultimate Load
Spun Pile	600	100	60	3148	250284	219295	333000	1574.119	1259.295	1885.17	159	174	177
Steel Pipe Pile	700	12	60	3794	250284	219295	333000	1897.0	1517.6	2271.9	132	145	147

Type of Foundation	Diameter [mm]	Allowable Single Pile Capacity [kN]			Spacing minimum of Pile	Xd	Efficiency	Number Of Pile AFTER efficiency		
		Service Load (Hydrostatic) SF=2	Service Load (Operating) sf=2.5	Ultimate Load sf=1.67				Service Load (Hydrostest)	Service Load (Operating)	Ultimate Load
Spun Pile	600	1574.119	1259.295	1885.17	2.6	4.3	0.79	201	222	224*
Stell Pipe Pile	700	1897.0	1517.6	2271.9	2.6	3.7	0.75	176	195	195*


BH-01
BH-02
0.00
-5.00
-10.00
-15.00
-20.00
-25.00
-30.00
-35.00
-40.00
-45.00
-50.00
-55.00
-60.00
-65.00
USCS Symbol : CH
Fat Clay
Consistency Stiff,
High Plasticity, Grey
N = 9 - 16
USCS Symbol : CH
Fat Clay
Consistency Soft,
High Plasticity, Grey
USCS Symbol : CH
Fat Clay
Consistency Stiff,
High Plasticity, Grey
N = 9 - 15
High Plastidty, Grey, N = 18 - 24
USCS Symbol : CH
Fat Clay
Consistency Very Hard,
High Plastidty, Grey
N = 50

Langkah Awal sebelum Mendesain adalah menentukan Parameter tanah

Field Soil Investigation

PT.LAPI ITB

LAPORAN HARIAN PEMBORAN

Project	Assesmen Tangki 946-TK-106	Tanggal	27-01-2020.	Lubang Bor : BH 02.
Klien	PT. Pertamina (persero) RU-II	Rencana kedalaman	60 m.	
Lokasi	TK-106 RU-II Sungai Pakning	Muka air tanah	P. 0.00. S. 0.00 Ful	
Elevasi		Cuaca	Mendung.	Halaman ke : I

Kedalaman		Waktu		Spesifikasi Pengeboran Core			Kondisi Tanah		Pemasangan Casing		
Dari (m)	sampai (m)	Jam	Durasi	Diameter (cm)	Panjang Core (m)	Persentase (%)			Kedalaman (m) Dari	sampai	Diameter Casing (cm)
0.00	0.50			73.	0.40	80.	Gambut.				
0.50	1.00			73.	0.50	100	Gambut.				
1.00	1.50			73	0.50	100	Tabung.	UDS. I			
1.50	1.95			2,5	0,45	100	Lempung lembek abu abu muda.	SPT			
1,95	3,00			73	0,95	90.	Lempung lembek abu abu muda.				
3,00	3,50			73	0.50	100	Tabung.	UDS 2.			
3,50	3,95			2,5	0.45	100	Lempung lembek abu abu muda.	SPT			
3.95	5.00			73	0.90	85	Lempung lembek abu abu muda.				
5.00	5.50			73	0,50	100	Tabung.	UDS. 3			
5,50	5,95			2,5	0,45	100.	Lempung lembek abu abu muda.	SPT			

Standart Penetration Test (SPT)

Kedalaman (m) Dari	sampai	N1	N2	N3	N2+N3
1.50	1.95	1/45			
3.50	3.95	1/45			
5.50	5.95	1/45			

Keterangan			
Moving mesin bor. Lanjut bor.	Master Bor	Agus.	
	Disiapkan oleh	Asep. S	
	Diperiksa oleh		
	Disetujui oleh		

PERTAMINA | PT.LAPI ITB

BORING LOG

STANDART :
Deep Boring Method Based On ASTM D-1452, D420, and D2113
SPT and Undisturbed Sampling Method Based on ASTM D1586 and D1587

PROJECT : SOIL INVESTIGATION TANGKI 946 TK-106 PT. PERTAMINA (PERSERO) RU-II SUNGAI PAKNING
CLIENT : PT.PERTAMINA (PERSERO
BORE HOLE : BH 01
COORDINATE : X : 9431.875 Y:10333.007 Z:93.627
ELEVATION : 10.933 m

GWL : 0 m
LOCATIO : PT.PERTAMINA (PERSERO) TK.106 RU-II SUNGAI PAKNING
SPT : AUTOMATIC HAMMER
DEPTH : 60 m

DATE : 17 Januari 2020
DRILLER : Agus
LOGGER
RECORDED BY

METHOD	DEPTH (M)	Sample	N1	N2	N3	N -	ROCK/SOIL DESCRIPTION
CORING AND SAMPLING	1.0						USCS Symbol : OL **Peat** Organic , Low Plasticity, Black
	2.0	UDS 1	1	0	0	N - 0	
	3.0						
	4.0	UDS 2	1	0	0	N - 0	
	5.0						
	6.0	UDS 3	1	0	0	N - 0	
	7.0						
	8.0	UDS 4	1	0	0	N - 0	
	9.0						
	10.0	UDS 5	1	0	0	N - 0	
	11.0						USCS Symbol : CL **Low Plasticity CLAY** Consistency very soft Clay, Low Plasticity, Light Grey
	12.0	UDS 6	1	0	0	N - 0	
	13.0						
	14.0	UDS 7	1	0	0	N - 0	
	15.0						
	16.0	UDS 8	1	0	0	N - 0	
	17.0						
	18.0	UDS 9	1	1	1	N - 2	
	19.0						
	20.0	UDS 10	1	1	2	N - 3	
...ING AND SAMPLING	21.0						
	22.0	UDS 11	1	2	2	N - 4	
	23.0						
	24.0	UDS 12	2	3	3	N - 6	USCS Symbol : CL **Low Plasticity CLAY** Consistency soft Clay, Low Plasticity, Light Grey
	25.0						
	26.0	UDS 13	1	1	2	N - 3	
	27.0						
	28.0	UDS 14	1	2	2	N - 4	
	29.0	UDS 15					

SPT BLOWS GRAPH: 10 20 30 40 50 60

VANE SHEAR METER: Kg/Cm²

PENETRO METER: Kg/Cm²

SAMPLE DOCUMENTATION

Laboratory Test

SOIL MECHANICS LABORATORY
FACULTY OF CIVIL AND ENVIRONMENTAL ENGINEERING
BANDUNG INSTITUTE OF TECHNOLOGY

LAB TEST RESUME

Form no. :
Issue/Revision :
Revision :

Date : March 3rd 2020

PROJECT : PT. LAPI ITB
LOCATION : Assement Tangki 946-TK-106 Area Kilang Sungai Pakning PT.Pertamina

No.	Bore Hole No. / Sample No.	Depth (m)	Sample Type	Gs	Density γ_m kN/m³	Density γ_d kN/m³	n	Sr %	Wn %	Atterberg Limits LL %	Atterberg Limits PL %	Atterberg Limits IP %	Class	PSD Gravel %	PSD Sand %	PSD Silt %	PSD Clay %	% finer by weight passing sieve # 200	Triaxial UU Total Stress c kN/m²	Triaxial UU Total Stress ϕ deg	Triaxial - CU Total Stress c kN/m²	Triaxial - CU Total Stress ϕ deg	Triaxial - CU Effective Stress c' kN/m²	Triaxial - CU Effective Stress ϕ' deg	Direct Shear c kN/m²	Direct Shear ϕ deg	Unconfined qu kN/m²	Unconfined cu kN/m²	Unconfined St	Consolidation e_o	Consolidation Cc
1	BH-01	1,00 - 1,50	UDS 1	2,22	12,80	5,22	0,77	100	145	165	31	134	CH	8	31	35	26	61	2,1	6,3	-	-	-	-	-	-	-	-	-	3,93	1,70
2		3,00 - 3,50	UDS 2	2,56	15,40	9,08	0,65	98	70	68	25	43	CH	0	3	33	64	97	0,4	1,7	-	-	-	-	-	-	-	-	-	2,45	0,73
3		5,00 - 5,50	UDS 3	2,54	13,80	6,78	0,73	96	103	103	25	78	CH	1	6	35	58	93	0,7	1,1	-	-	-	-	-	-	-	-	-	2,97	1,24
4		7,00 - 7,50	UDS 4	2,56	14,40	7,51	0,71	98	92	90	27	64	CH	0	0	32	68	100	5,5	0,7	-	-	-	-	-	-	-	-	-	2,42	1,21
5		9,00 - 9,50	UDS 5	2,59	15,20	8,53	0,67	99	78	77	22	55	CH	0	0	37	63	100	1,9	3,1	-	-	-	-	-	-	-	-	-	1,86	0,69
6		11,00 - 11,50	UDS 6	2,56	14,10	7,34	0,71	95	92	91	23	68	CH	0	0	38	62	100	2,4	2,0	-	-	-	-	-	-	-	-	-	2,33	1,44
7		13,00 - 13,50	UDS 7	2,58	14,20	7,26	0,72	97	95	93	28	64	CH	0	0	22	78	100	5,8	0,8	-	-	-	-	-	-	-	-	-	2,53	1,29
8		15,00 - 15,50	UDS 8	2,58	15,10	8,48	0,67	98	79	85	25	60	CH	0	0	28	72	100	5,0	1,7	-	-	-	-	-	-	-	-	-	2,16	1,10
9		17,00 - 17,50	UDS 9	2,57	14,30	7,36	0,71	98	95	97	29	68	CH	0	0	15	85	100	7,9	3,3	-	-	-	-	-	-	-	-	-	2,33	1,22
10		19,00 - 19,50	UDS 10	2,62	16,60	10,75	0,59	99	54	71	23	48	CH	0	2	24	73	98	11,3	1,9	-	-	-	-	-	-	-	-	-	1,64	0,72
11		21,00 - 21,50	UDS 11	2,64	15,60	9,46	0,64	99	67	74	24	49	CH	0	1	27	72	99	6,4	0,8	-	-	-	-	-	-	-	-	-	1,77	0,67
12		23,00 - 23,50	UDS 12	2,63	15,70	9,37	0,64	99	67	77	26	50	CH	0	0	24	75	100	14,7	1,2	-	-	-	-	-	-	-	-	-	1,74	0,76
13		25,00 - 25,50	UDS 13	2,65	15,40	8,84	0,67	99	74	83	25	58	CH	0	0	20	79	100	13,0	5,8	-	-	-	-	-	-	-	-	-	1,72	0,74
14		27,00 - 27,50	UDS 14	2,60	16,10	10,03	0,61	99	61	78	26	52	CH	0	0	24	76	100	10,3	1,4	-	-	-	-	-	-	-	-	-	1,88	0,90
15		29,00 - 29,50	UDS 15	2,59	15,90	9,75	0,62	98	63	80	27	53	CH	0	0	20	80	100	9,0	0,8	-	-	-	-	-	-	-	-	-	1,82	0,79
TOTAL NO. OF TESTS				15	15	15				15				15					15		-				-		-			15	

Notes:

US = Undisturbed Sample
Gs = Specific gravity
γ_m = Bulk Density
γ_d = Dry Density
Wn = Moisture Content

LL = Liquid Limit
PL = Plastic Limit
IP = Plasticity Index
c = Cohesion
ϕ = Friction Angle

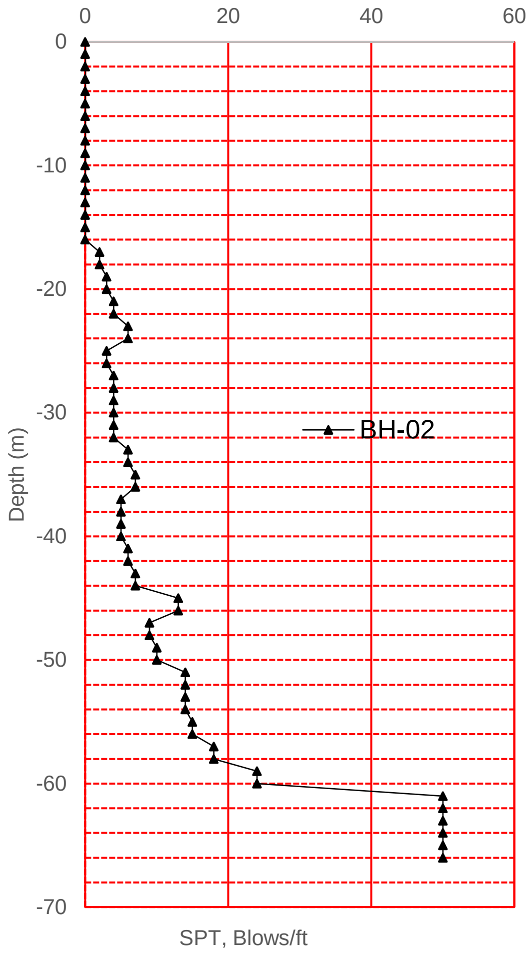
UU = Unconsolidated Undrained
CU = Consolidated Undrained
Cc = Compression Index
UCS = Unconfined Compression Strength
qu = Ultimate Stress

Checked by Engineer
Date : 3 Maret 2020
Name and Signature

Faisal Mirza
u.n. Hasnaa

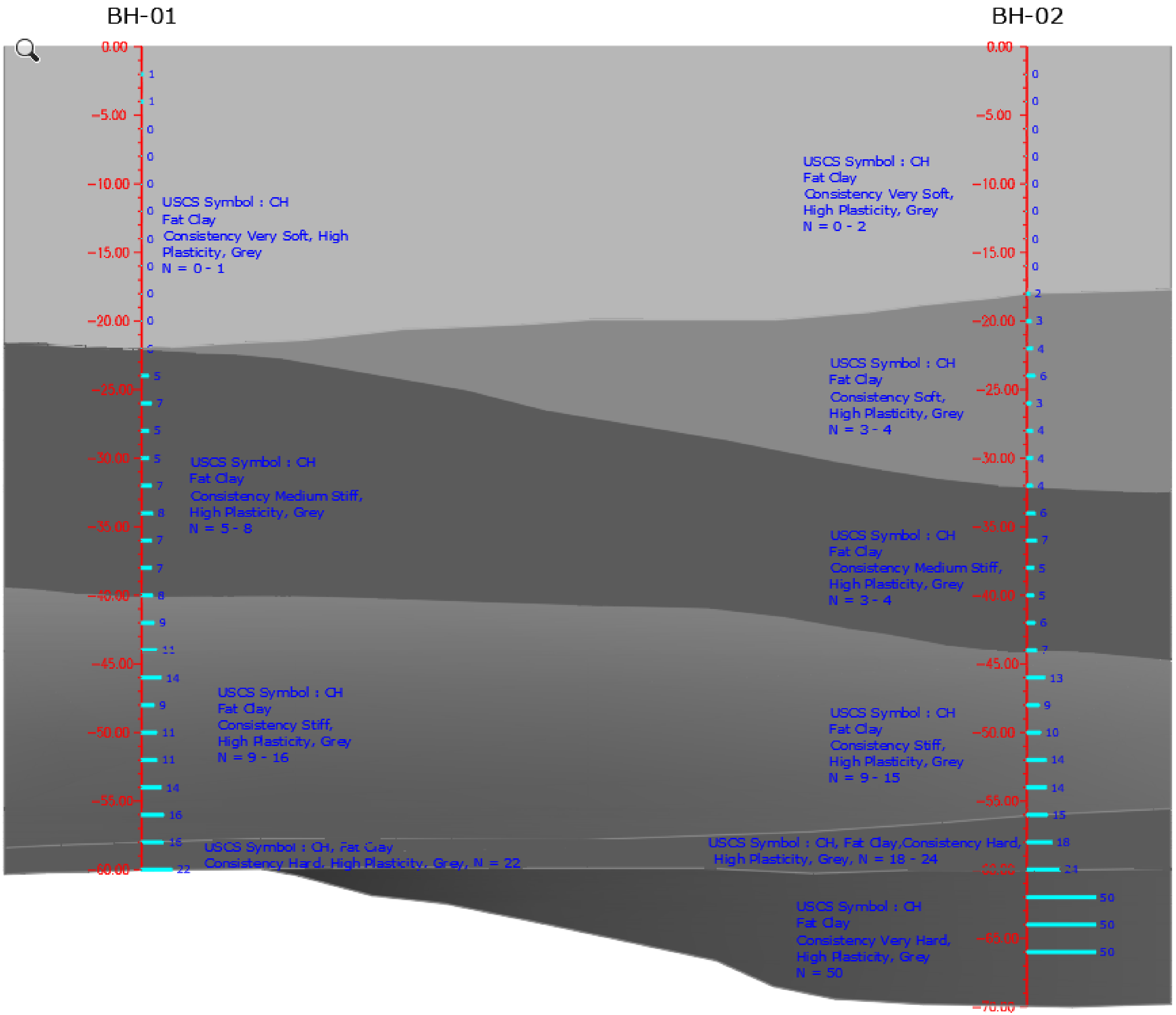
Page 1 of 2

Grafik NSPT

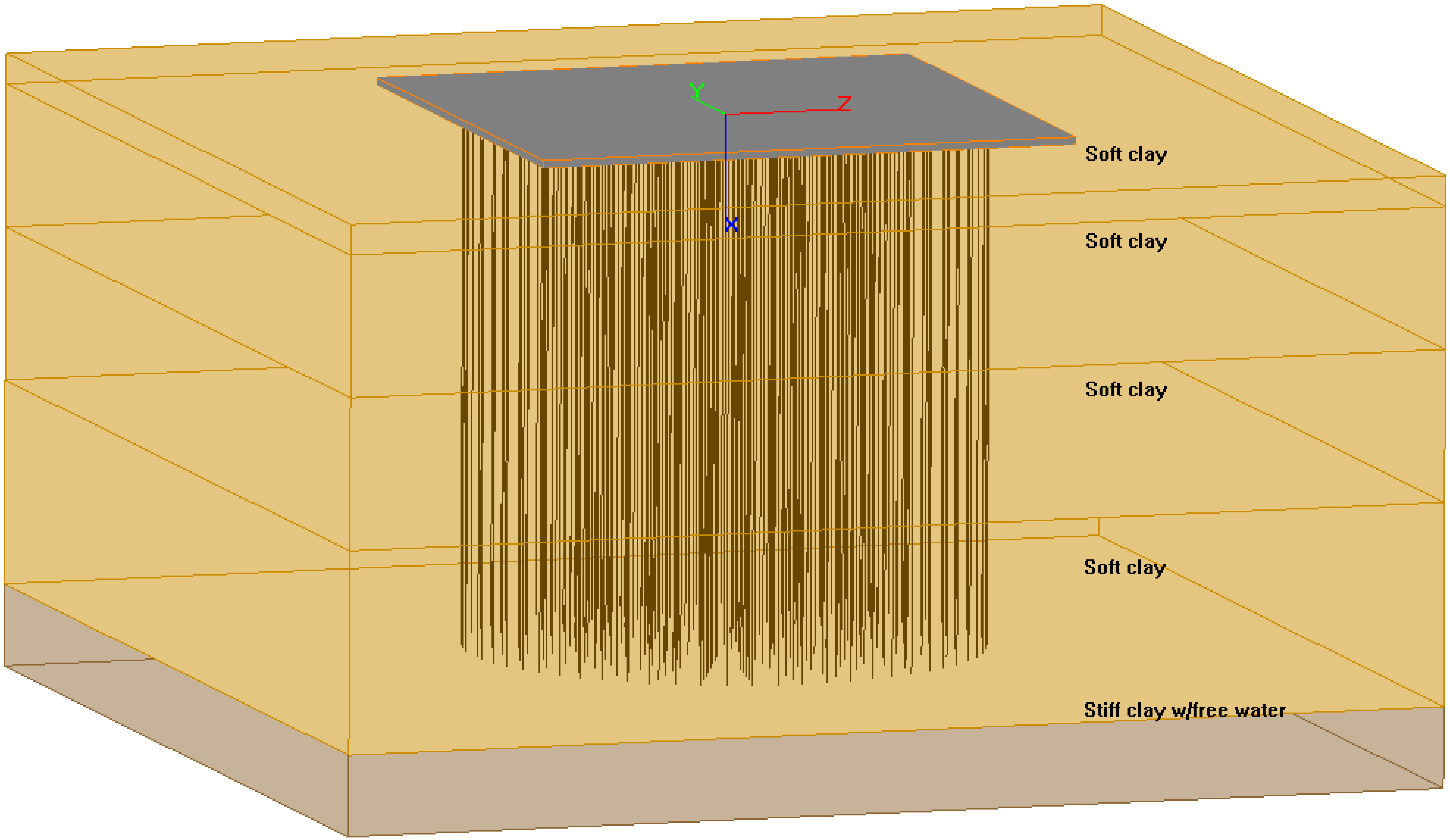


Desain Parameter tanah

No.	Jenis Tanah	Kedalaman (m)	NSPT (avg)	γ , (kN/m³)	Kohesif (cu) (kN/m2)	Friction Angle (ϕ)	E50 (kN/m2)
1	Organic Clay	00.00 – 03.00	1	4.8	6	0	0.02
2	Very Soft Clay	03.00 – 17.00	1	7	6	0	0.02
3	Soft Clay	17.00 – 32.00	4	9.2	22	0	0.02
4	Soft to Medium Clay	32.00 – 52.00	8	8.2	49	0	0.01
5	Stiff Clay	52.00 - 60.00	24	9	96	0	0.005
6	Hard Clay	60.00-66.00	50	19	144	0	0.004



BH-01
BH-02
USCS Symbol : CH
Fat Clay
Consistency Very Soft, High Plasticity, Grey
N = 0 - 1
USCS Symbol : CH
Fat Clay
Consistency Medium Stiff, High Plasticity, Grey
N = 5 - 8
USCS Symbol : CH
Fat Clay
Consistency Stiff, High Plasticity, Grey
N = 9 - 16
USCS Symbol : CH, Fat Clay
Consistency Hard, High Plasticity, Grey, N = 22
USCS Symbol : CH
Fat Clay
Consistency Very Soft, High Plasticity, Grey
N = 0 - 2
USCS Symbol : CH
Fat Clay
Consistency Soft, High Plasticity, Grey
N = 3 - 4
USCS Symbol : CH
Fat Clay
Consistency Medium Stiff, High Plasticity, Grey
N = 3 - 4
USCS Symbol : CH
Fat Clay
Consistency Stiff, High Plasticity, Grey
N = 9 - 15
USCS Symbol : CH, Fat Clay, Consistency Hard, High Plasticity, Grey, N = 18 - 24
USCS Symbol : CH
Fat Clay
Consistency Very Hard, High Plasticity, Grey
N = 50


Y
Z
X
Soft clay
Soft clay
Soft clay
Soft clay
Stiff clay w/free water

Introduction

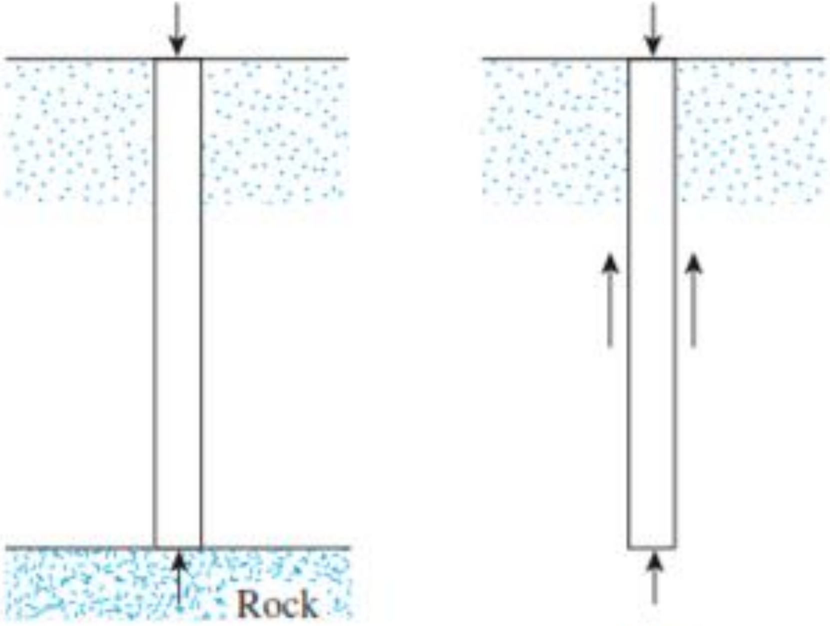
Piles are structural members that are made of steel, concrete, or timber. They are used to build pile foundations, which are deep and which cost more than shallow foundations

Introduction

The following list identifies some of the conditions that require pile foundations (Vesic, 1977):

a) Kondisi lapisan tanah atas memiliki kompressibilitas tinggi dan kurang kuat memikul distribusi beban dari struktur atas. Pile berfungsi untuk mendistribusikan beban ke lapisan batuan atau tanah yang lebih kuat.

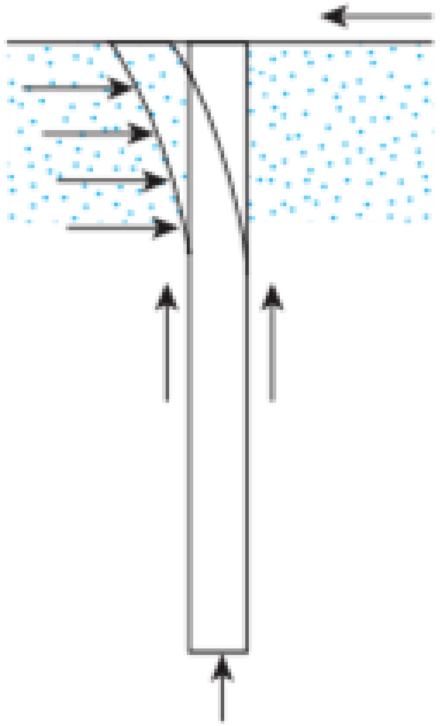
b) Jika lapisan tanah keras cukup jauh dari permukaan tanah. Pile dapat digunakan untuk mentransfer beban struktur ke tanah secara bertahap. Kemampuan tiang menahan beban berasal dari tahanan friksi yang terbentuk dari interface antara tiang+tanah.



(a) (b)

Introduction

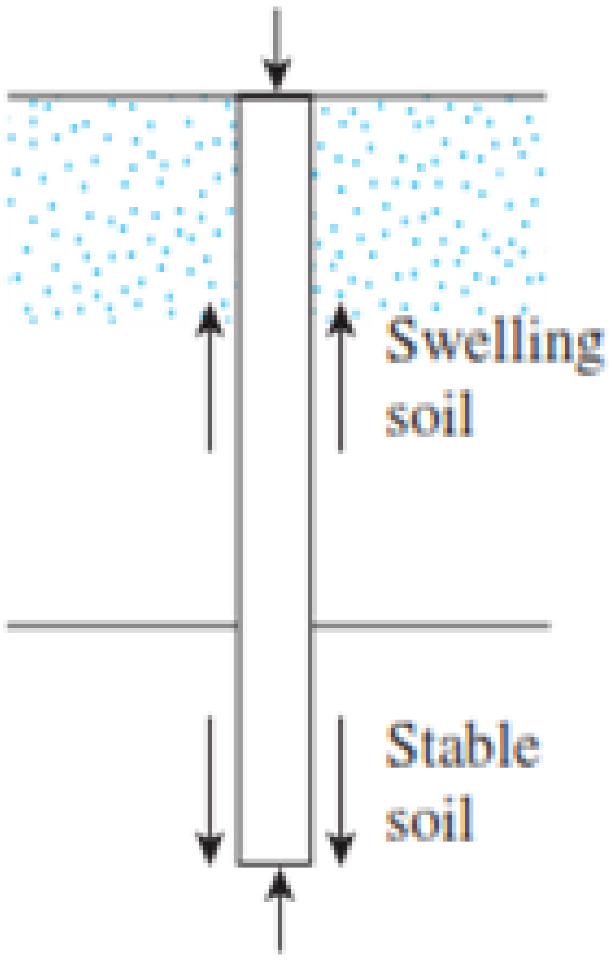
c) Tiang mampu menahan tekuk (*resist to bending*) di saat yang sama juga menahan beban vertical dari struktur atas. Kondisi ini terjadi saat terdapat struktur sangat tinggi yang memiliki dampak signifikan terhadap beban gempa dan beban angin



(c)

Introduction

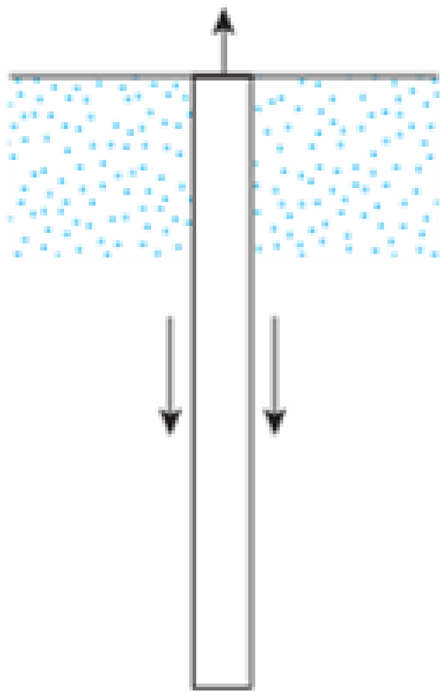
d) Kondisi berada di tanah yang mudah runtuh atau tanah ekspansif (mengembang). Penggunaan pile menjadi alternatif untuk kondisi zona aktif ini



(d)

Introduction

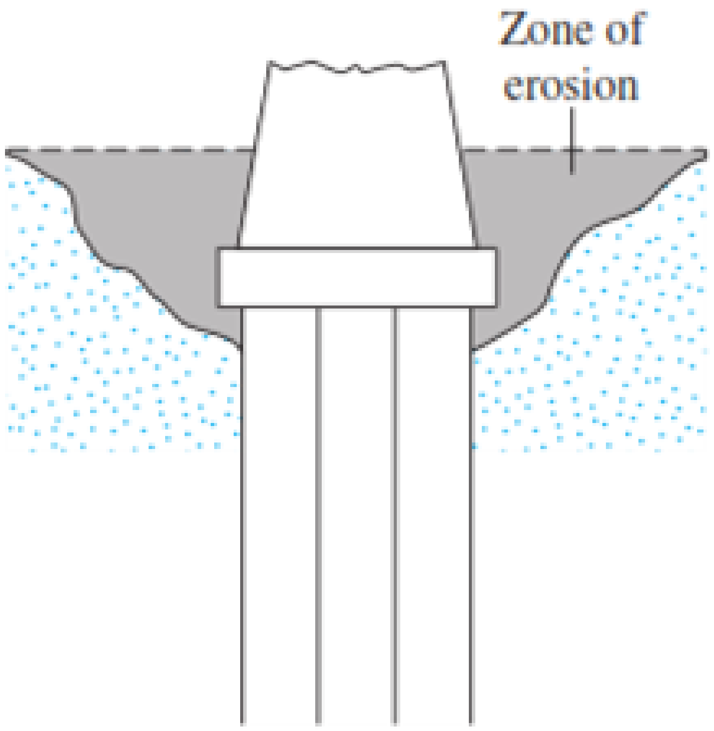
e) Pile juga digunakan untuk menahan beban *uplift* untuk beberapa struktur seperti tower transmisi, platform offshore, dan alas basement yang berada dibawah permukaan air tanah



(e)

Introduction

f) Abutment jembatan dan piers biasanya juga menggunakan *pile foundation* untuk menghindari hilangnya kapasitas daya dukung



(f)

Pile Foundation

Type of Piles and Their Structural Characteristics

Different types of piles are used in construction work, depending on the type of load to be carried, the subsoil conditions, and the location of the water tables.

Piles can be divided into the following categories:

(a) Steel piles
(b) Concrete piles
(c) Wooden (timber) piles
(d) Composite piles.

Type of Piles and Their Structural Characteristics

Steel Pile

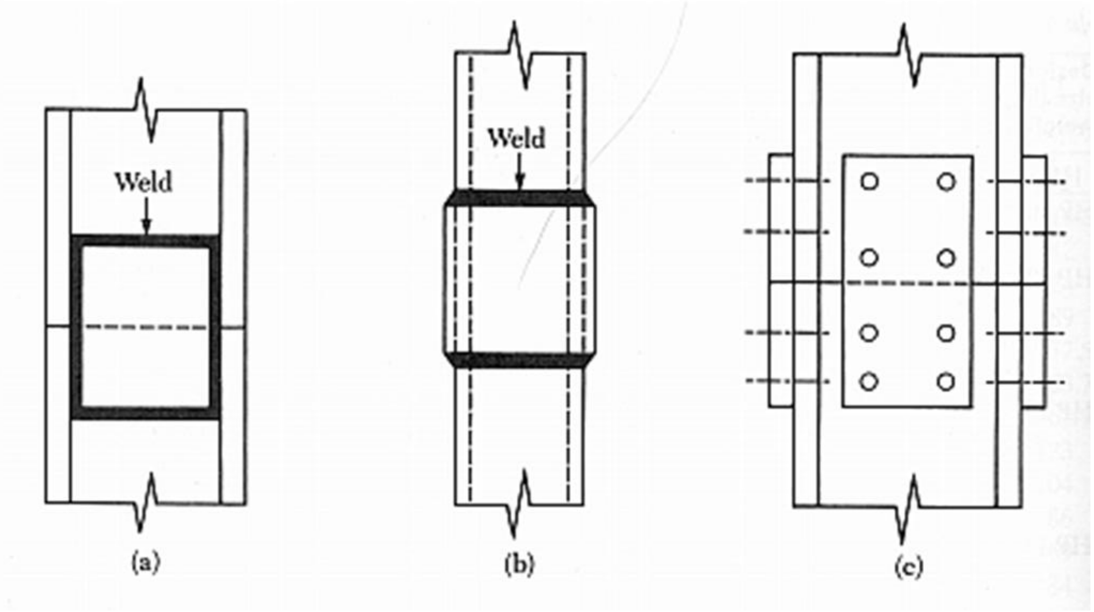
Steel piles generally are either pipe piles or rolled H-section piles. Pipe piles can be driven into the ground with their ends open or closed. In many cases, the pipe piles are filled with concrete after they have been driven.

Type of Piles and Their Structural Characteristics

Steel Pile

When necessary, steel piles are spliced by welding or by riveting.

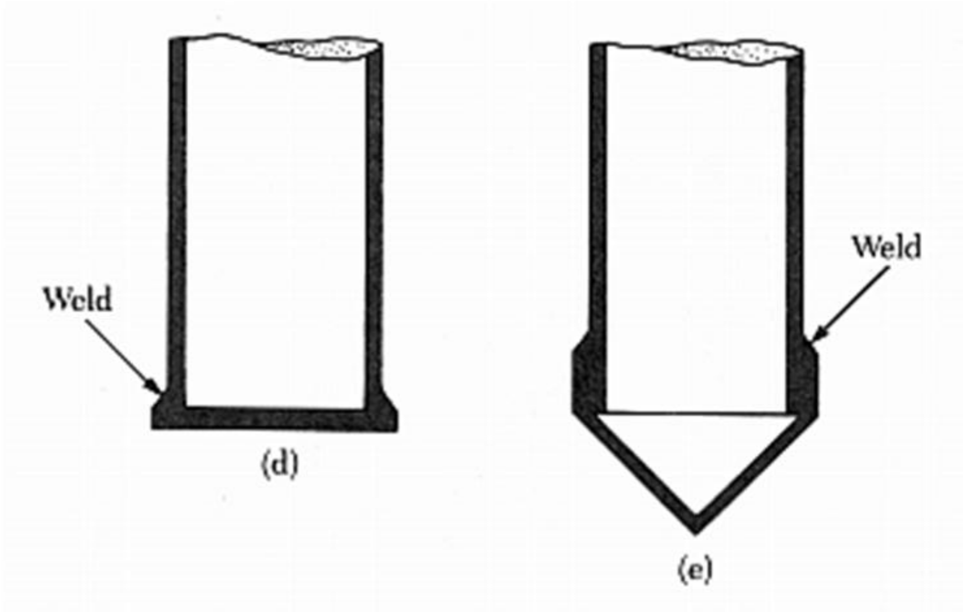
a) Figure shows a typical splice by welding for an H-pile
b) Figure shows b typical splice by welding for a pile piles.
c) Figure shows c diagram of splice of an H-pile by rivets or bolts.



Type of Piles and Their Structural Characteristics

Steel Pile

When hard driving conditions are expected, such as driving through dense gravel, shale, or soft rock, steel piles can be fitted with driving points or shoes.
Figures (d) and (e) shows diagram of two types of shoe for pipe piles.



Type of Piles and Their Structural Characteristics

Steel Pile

Steel Pile dapat mengalami korosif misalnya akibat rawa, gambut, dan tanah organik lainnya yang bersifat korosif. Untuk mengatasi ini biasanya direkomendasikan untuk menambah ketebalan baja (melebihi ketebalan penampang yang didesain). Pada keadaan lain dari pabriknya sudah diberi lapisan *epoxy* yang cukup baik untuk menahan korosif, yang mana lapisan ini juga tidak mudah rusak saat dipancang. Selain itu alternatif lain dapat membungkus baja dengan beton di zona korosifnya.

Type of Piles and Their Structural Characteristics

Steel Pile

<u>Kelebihan :</u>

1. Mudah dipotong atau ditambah ukuran panjang tiang
2. Bisa berdiri pada kondisi tegangan yang besar akibat pemancangan
3. Bisa menembus lapisan keras seperti kerikil padat dan batuan lunak
4. Mampu memikul kapasitas beban yang besar

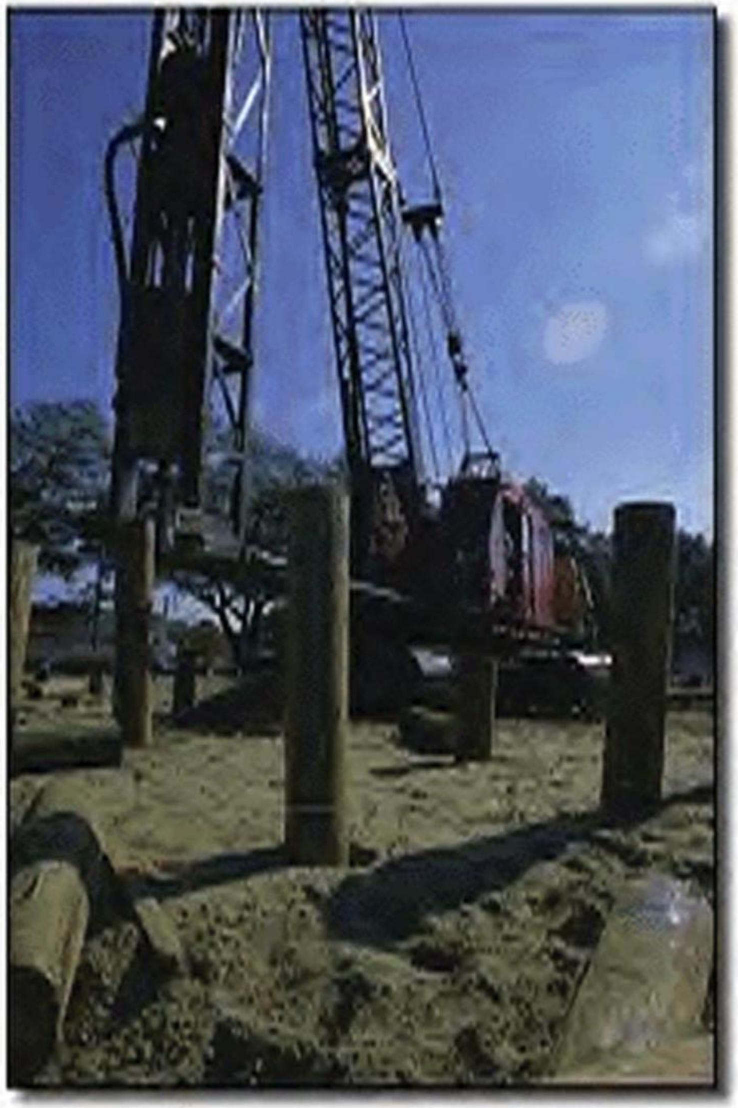
<u>Kekurangan :</u>

1. Relatif mahal
2. Menimbulkan kebisingan
3. Rentan korosif
4. H-piles bisa mengalami defleksi akibat pemancangan pada lapisan keras

Type of Piles and Their Structural Characteristics

Concrete Pile

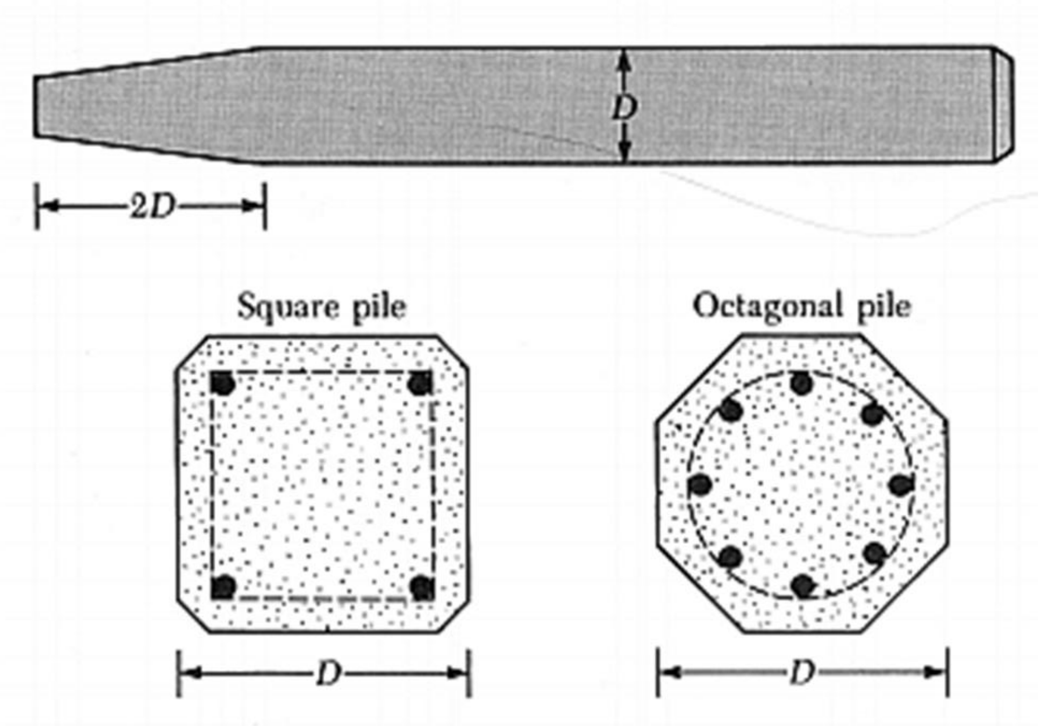
Concrete piles may be divided into two basic categories:
(a) precast piles
and (b) cast-in-situ piles.



Type of Piles and Their Structural Characteristics

Precast Concrete Pile (Tiang Pancang Beton)

Precast piles adalah pile yang sudah disiapkan (Pile fabrikasi) dengan besar perkuatan sudah ditentukan pada umumnya, penampang dapat berupa segiempat atau oktagonal.



Keuntungan :

a. Bisa untuk pemancangan pada tanah keras
b. Tahan korosif
c. Mudah dikombinasikan dengan struktur beton lainnya

Kerugian :

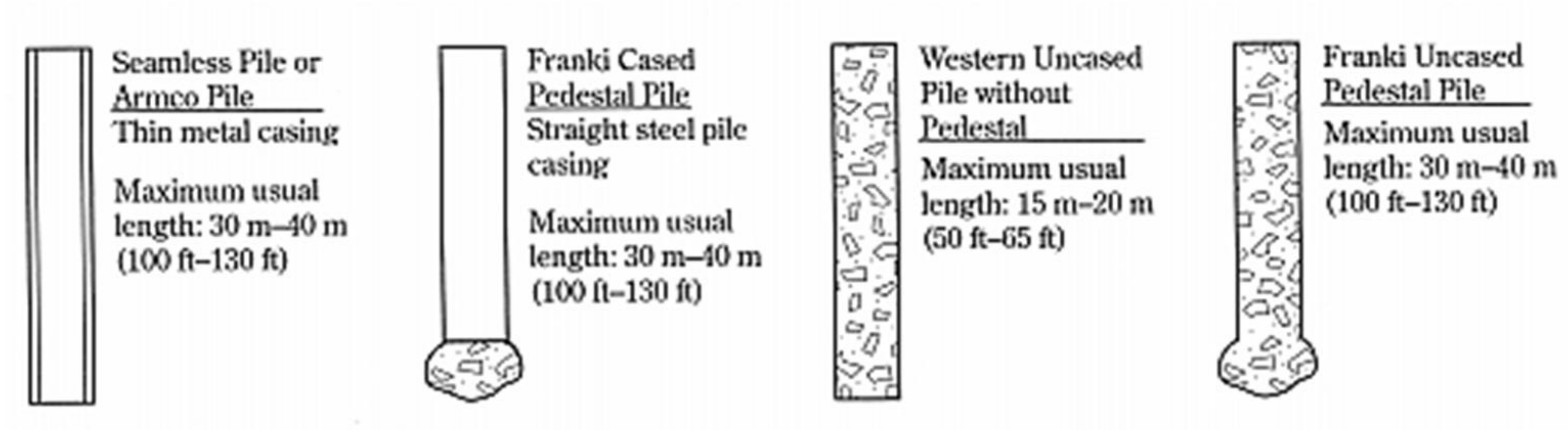
a. Sulit untuk dilakukan pemotongan
b. Transportasi sulit

Type of Piles and Their Structural Characteristics

Cast In situ Concrete Pile (Bored pile)

Cast-in-situ atau *cast-in-place*, adalah pile yang dibentuk dengan membuat lubang pada tanah, dilakukan pemasangan tulangan lalu diisi dengan beton. Pile ini dibagi dalam dua kategori yaitu :

1. pile dengan casing (**cased**) dan 2. pile tanpa casing (**uncased**)



Type of Piles and Their Structural Characteristics

Cased Cast In situ Concrete Pile

Pile dengan casing dibuat dengan memancang casing baja ke dalam tanah. Saat pile sudah mencapai kedalaman yang mencukupi, casing diisi dengan beton (bisa sebelumnya ditambah tulangan)

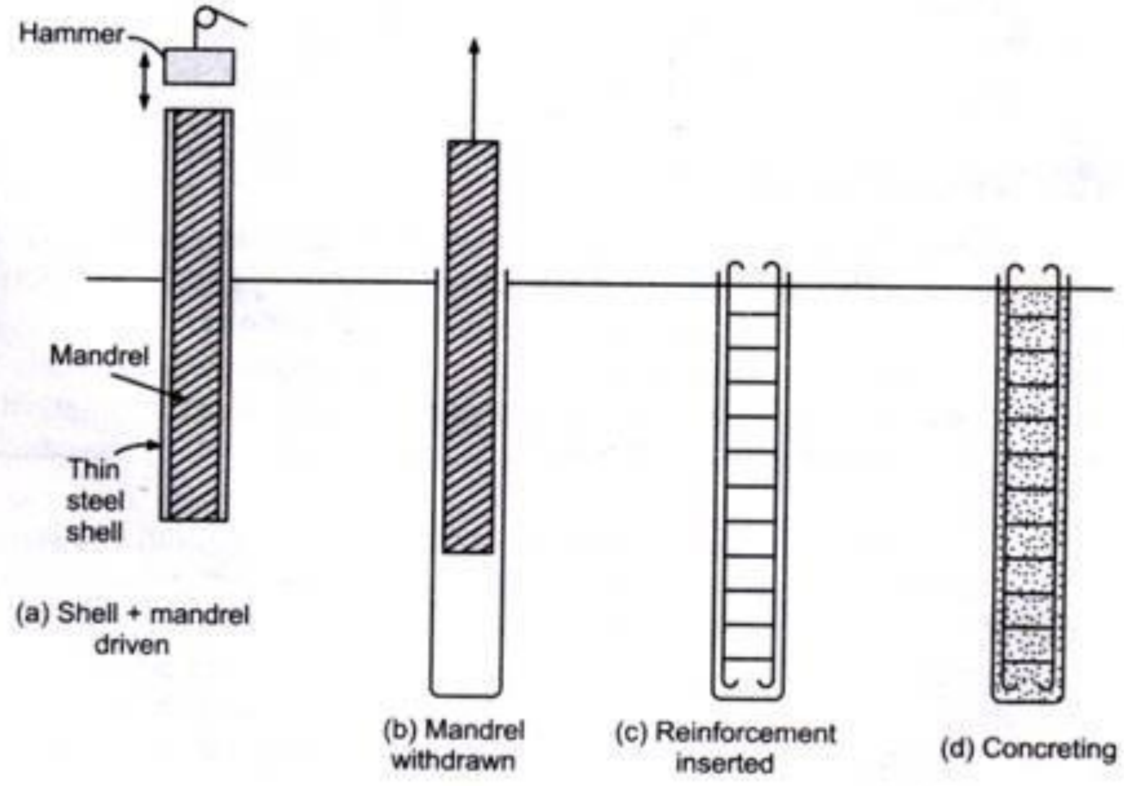


FIG. 11.28 Cased driven cast-in-situ piles

Type of Piles and Their Structural Characteristics

Cased Cast In situ Concrete Pile

Keuntungan :

1. Relatif murah
2. Kondisi Tanah bisa diselidiki setelah pemasangan casing dan sebelum pengisian beton
3. Mudah untuk diperpanjang

Kelemahan :

1. Sulit disambung jika telah dicor beton
2. Casing rentan mengalami kerusakan selama pemancangan

Type of Piles and Their Structural Characteristics

UnCased Cast In situ Concrete Pile

Uncased Pile dibuat dengan memancang casing baja pada kedalaman yang telah ditentukan, saat dituangkan beton segar, casing ditarik secara bertahap

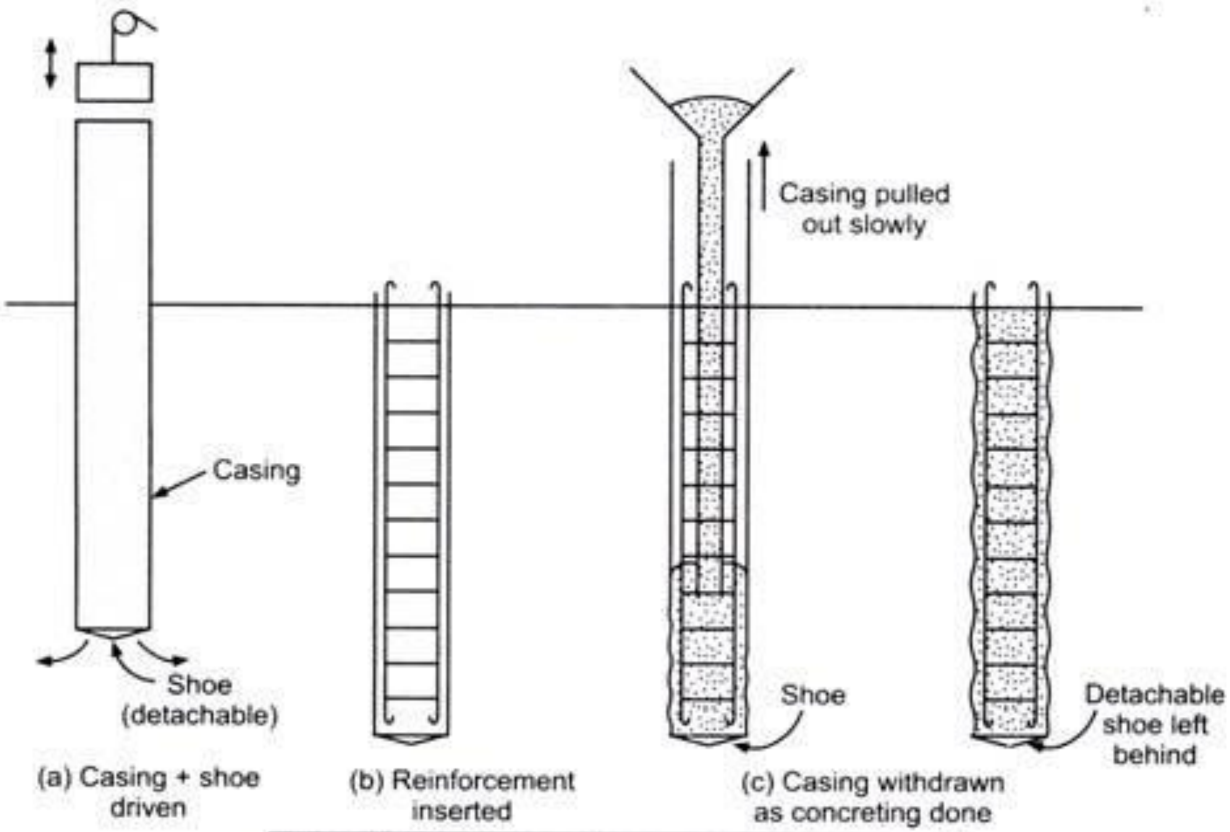


FIG. 11.29 Uncased driven cast-in-situ piles

Type of Piles and Their Structural Characteristics

UnCased Cast In situ Concrete Pile

Keuntungan :

1. Pada awalnya lebih ekonomis
2. Bisa diselesaikan pada berbagai elevasi kedalaman

Kelemahan :

1. Pori bisa tercipta jika pengisian beton terlalu cepat
2. Sulit disambung jika telah dibeton
3. Pada tanah lunak, beberapa kasus sisi lubang dapat runtuh.
4. Pada kondisi tertentu, terdapat rongga besar dalam tanah (berupa lensa tanah), hal ini merugikan karena harus diisi beton terus menerus hingga menutupi lensa tersebut

Type of Piles and Their Structural Characteristics

Timber Pile

Timber pile adalah tiang dari kayu, maximal Panjang tiang hanya 10-20 m. Kualitas kayu yang digunakan sebagai timber pile harus lurus, mulus, tanpa cacat. Timber pile tidak bisa menahan tegangan pemancangan terlalu besar sehingga kapasitas tiang cukup terbatas. Beberapa kasus diberikan sepatu baja (*steel shoes*) untuk menghindari kerusakan pada ujung tiang

Kepala atau bagian atas *timber piles* juga bisa mengalami kerusakan selama pemancangan. Untuk menghindari hal ini bagian atas tiang diberikan pengikat berbahan metal atau topi tiang

Type of Piles and Their Structural Characteristics

Composite Pile

Porsi bagian atas atau bawah tiang komposit dibuat dari material yang berbeda. Contohnya, tiang komposit dapat berupa baja + beton atau kayu+beton. Untuk tiang komposit baja+beton terdiri atas porsi lebih rendah baja dan porsi lebih atas *cast in place* concrete.

Estimating Pile Length

Pile terbagi atas tiga kategori berdasarkan panjang dan mekanisme transfer beban di dalam tanah.

1. Point bearing piles
2. Friction piles
3. Compaction piles

Estimating Pile Length

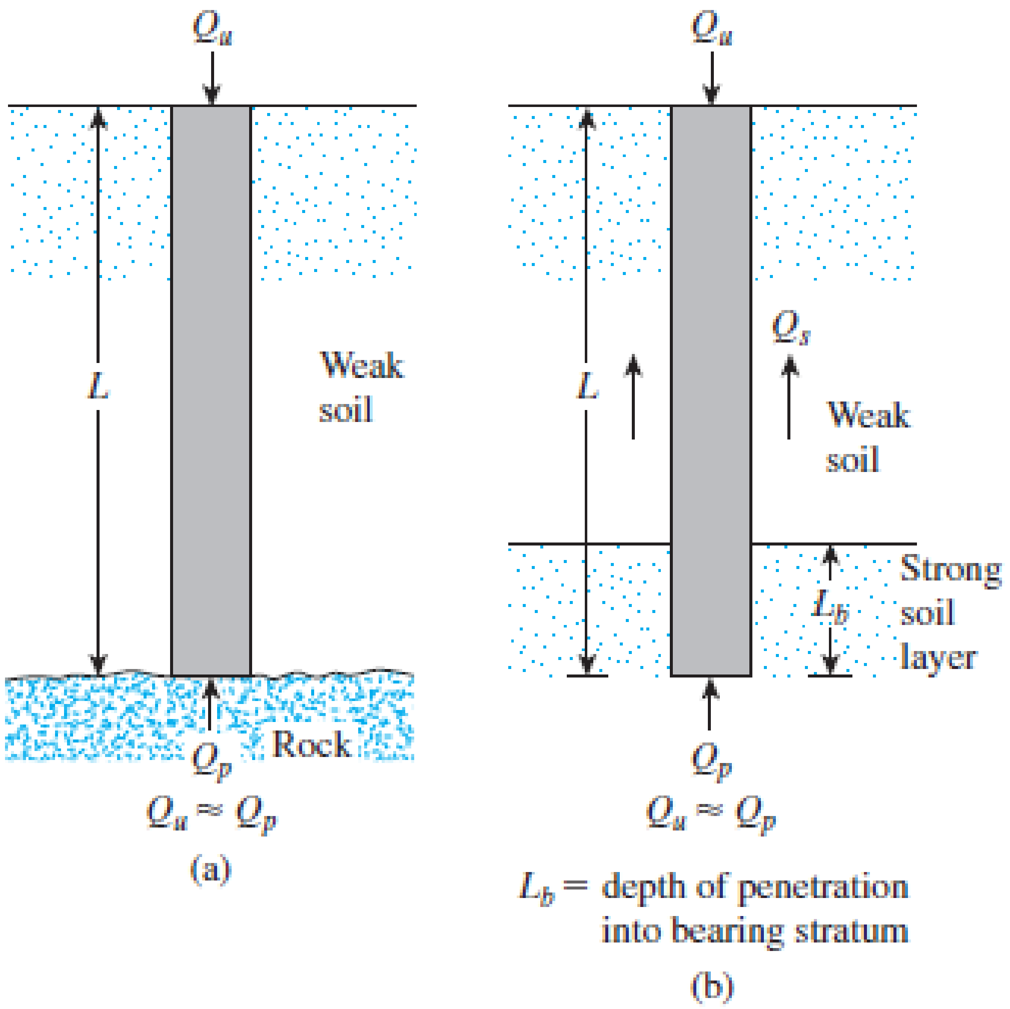


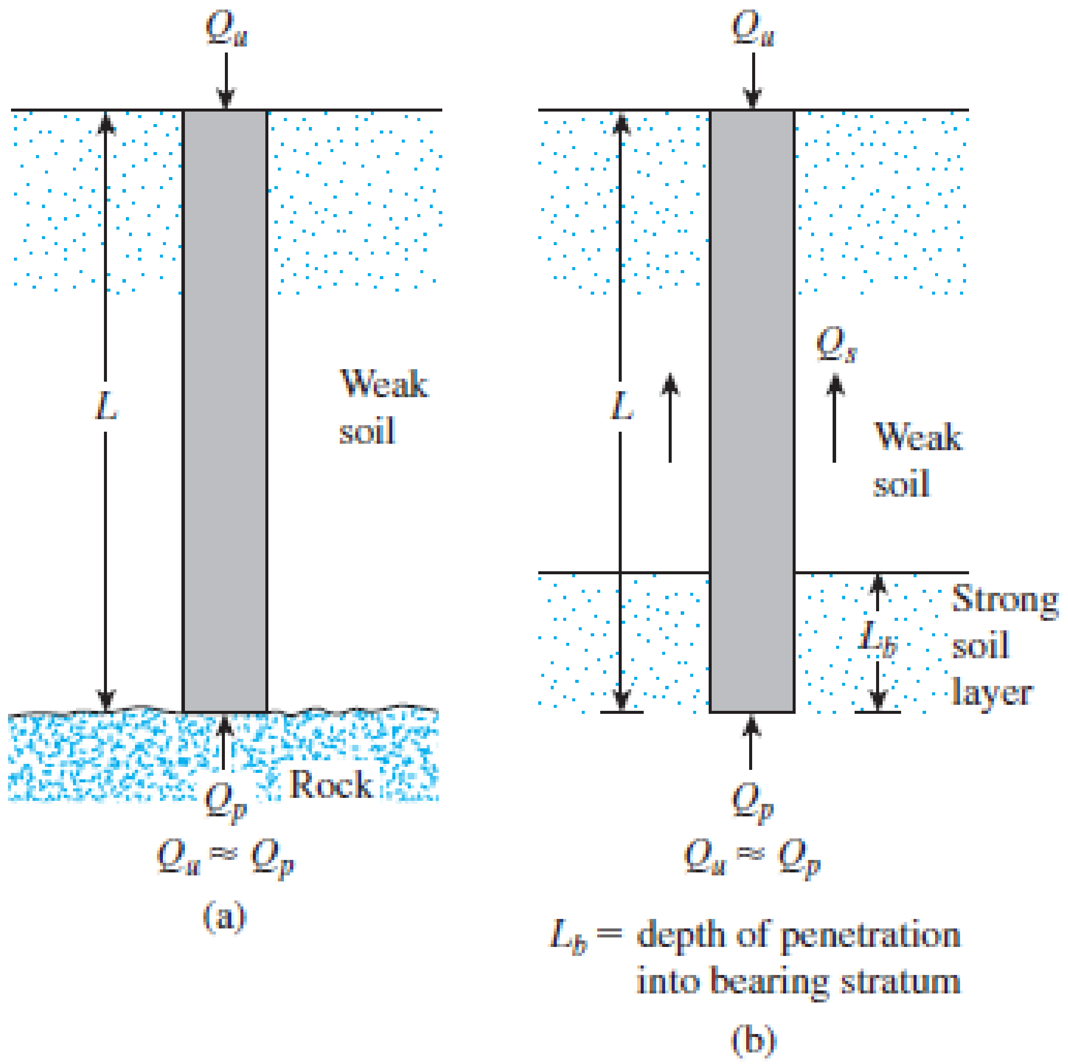
Figure 9.6 (a) and (b) Point bearing piles; (c) friction piles

$$Q_u = Q_p + Q_s$$

where

Q_p = load carried at the pile point
Q_s = load carried by skin friction developed at the side of the pile (caused by shearing resistance between the soil and the pile)

Estimating Pile Length



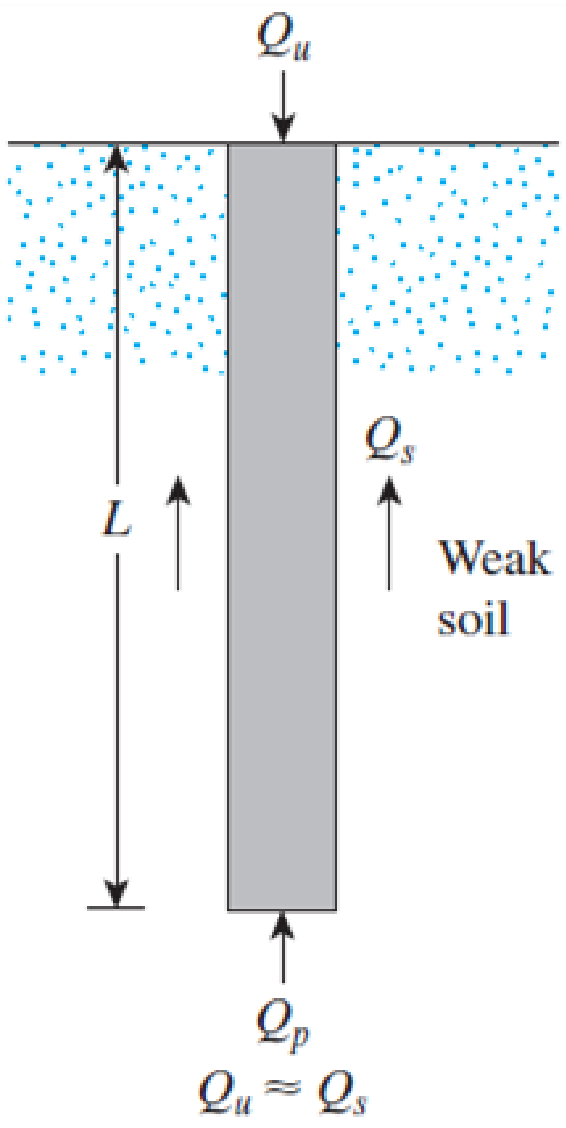
Point bearing pile

Kapasitas tiang bergantung sepenuhnya pada kapasitas daya dukung material tanah dasar di ujung tiang yang disebut *point bearing pile.* Dalam banyak kasus Panjang tiang mencukupi hingga sampai tanah dasar

If Q_s is very small,

$$Q_s \approx Q_p$$

Estimating Pile Length



(c)

Friction piles

Kondisi saat tidak ada lapisan tanah keras di dasar ujung tiang sehingga pile didesain sangat panjang dan tidak ekonomis. Pada tipe subsoil ini, tiang dipancang menembus tanah lunak hingga kedalaman tertentu. Daya dukung ijin tiang dapat di lihat pada rumus di bawah ini. Dimana nilai daya dukung ujung Qp relative kecil.

$$Q_u \approx Q_s$$

Estimating Pile Length

Compaction piles

Under certain circumstances, piles are driven in granular soils to achieve proper compaction of soil close to the ground surface. These piles are called *compaction piles*. The lengths of compaction piles depend on factors such as (a) the relative density of the soil before compaction, (b) the desired relative density of the soil after compaction, and (c) the required depth of compaction. These piles are generally short; however, some field tests are necessary to determine a reasonable length.

Equations for Estimating Pile Capacity

The ultimate load-carrying capacity Q_u of a pile is given by the equation

$$Q_u = Q_p + Q_s \tag{9.9}$$

where

Q_p = load-carrying capacity of the pile point
Q_s = frictional resistance (skin friction) derived from the soil–pile interface (see Figure 9.11)

$$Q_{all} = \frac{Q_u}{FS}$$

where

Q_{all} = allowable load-carrying capacity for each pile
FS = factor of safety

The factor of safety generally used ranges from 2.5 to 4, depending on the uncertainties surrounding the calculation of ultimate load.

Equations for Estimating Pile Capacity

Kapasitas daya dukung = Luas area × daya dukung per unit

$Q_p = A_p \times q_p$ Daya dukung ujung (Point bearing capacity)

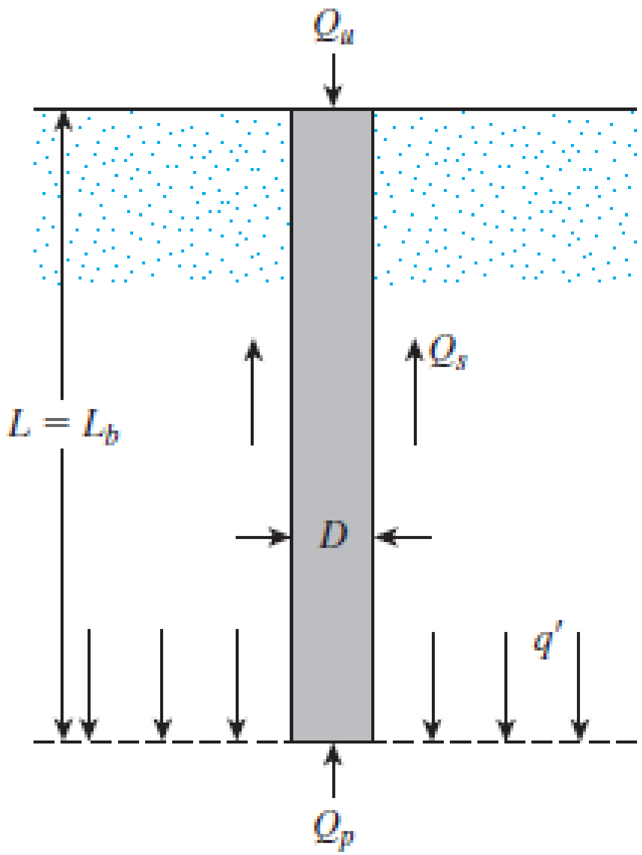
A_p = luas penampang pile

$Q_s = A_s \times q_s$ Daya dukung tahanan friksi (friction resistance)

A_s = luas selimut pile

$Q_u = Q_p + Q_s$ Daya dukung ultimate

$Q_{all} = \frac{Q_u}{FS}$ Daya dukung ijin



L = length of embedment
L_b = length of embedment in bearing stratum

(a)

Equations for Estimating Pile Capacity

Point bearing capacity, Qp

$Q_p = A_p \times q_p$ Daya dukung ujung (Point bearing capacity)

A_p = luas penampang pile

$$q_u = q_p = c'N_c^* + qN_q^* + \cancel{\gamma D N}{}_\gamma^*$$

Kohesif tanah

Tegangan Vertikal efektif

Dimensi tiang

Equations for Estimating Pile Capacity

The ultimate load-carrying capacity Q_u of a pile is given by the equation

$$Q_u = Q_p + Q_s \tag{9.9}$$

where

Q_p = load-carrying capacity of the pile point
Q_s = frictional resistance (skin friction) derived from the soil–pile interface (see Figure 9.11)

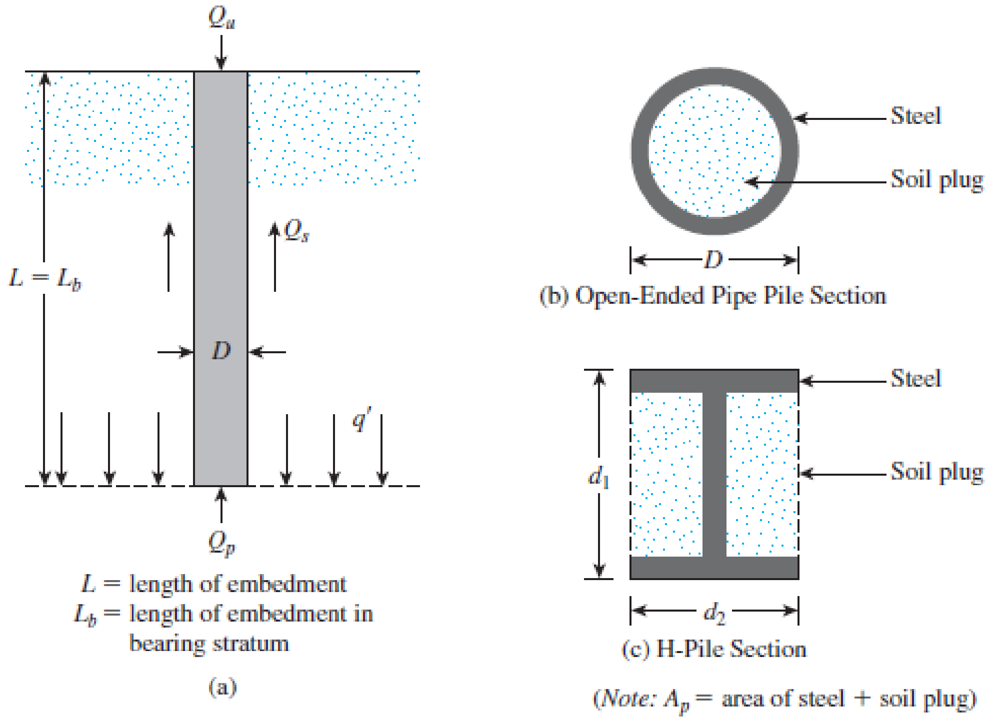


Figure 9.11 Ultimate load-carrying capacity of pile

Equations for Estimating Pile Capacity

Point bearing Capacity, Qp

$$Q_p = A_p q_p = A_p(c'N_c^* + q'N_q^*)$$

where

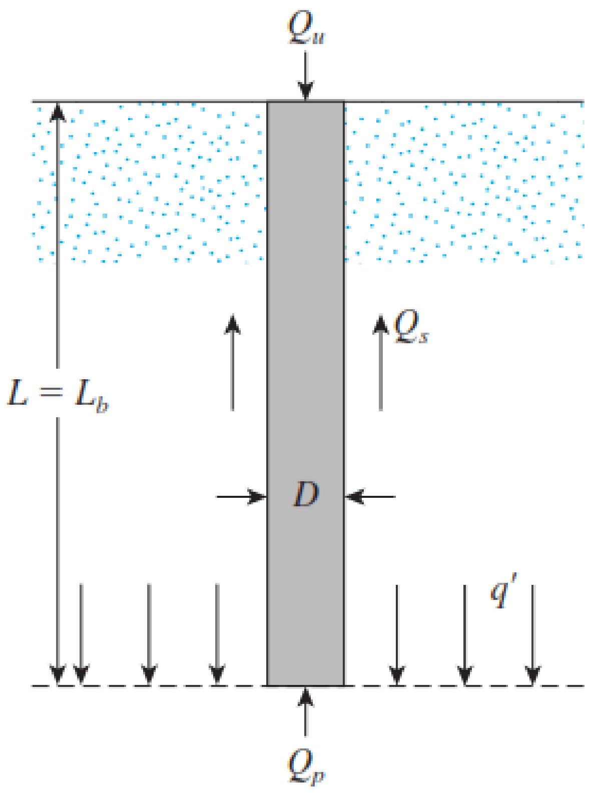
A_p = area of pile tip
c' = cohesion of the soil supporting the pile tip
q_p = unit point resistance
q' = effective vertical stress at the level of the pile tip
N_c^*, N_q^* = the bearing capacity factors

Frictional Resistance, Qs

$$Q_s = \Sigma p\, \Delta L f$$

where

p = perimeter of the pile section
ΔL = incremental pile length over which p and f are taken to be constant
f = unit friction resistance at any depth z



L = length of embedment
L_b = length of embedment in bearing stratum

Estimating Point Bearing Capacity, Qp

Estimating Point Bearing Capacity, Qp

Meyerhof's method for estimating Qp in **SAND**

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

$$q_l = 0.5\, p_a N_q^* \tan \phi'$$

where

p_a = atmospheric pressure (=100 kN/m^2 or 2000 lb/ft^2)
ϕ' = effective soil friction angle of the bearing stratum

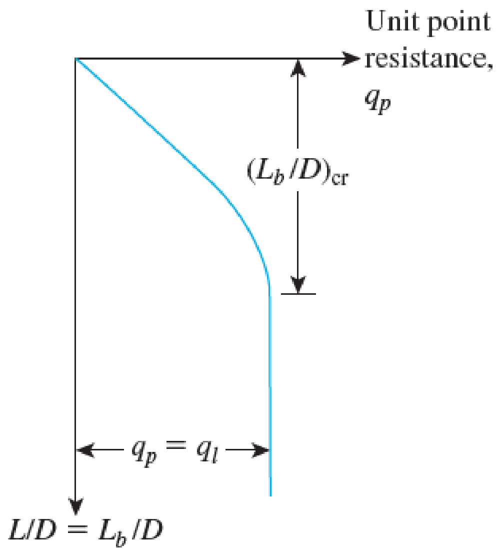


Figure 9.12 Nature of variation of unit point resistance in a homogeneous sand

The point bearing capacity, q_p, of a pile in sand generally increases with the depth of embedment in the bearing stratum and reaches a maximum value at an embedment ratio of $L_b/D = (L_b/D)_{cr}$. Note that in a homogeneous soil L_b is equal to the actual embedment length of the pile, L. However, where a pile has penetrated into a bearing stratum, $L_b < L$. Beyond the critical embedment ratio, $(L_b/D)_{cr}$, the value of q_p remains constant ($q_p = q_l$). That is, as shown in Figure 9.12 for the case of a homogeneous soil, $L = L_b$.

Table 9.5 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

Estimating Point Bearing Capacity, Qp

Meyerhof's method for estimating Qp in **SAND**

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

$$q_l = 0.5\, p_a N_q^* \tan \phi'$$

where

p_a = atmospheric pressure (=100 kN/m^2 or 2000 lb/ft^2)
ϕ' = effective soil friction angle of the bearing stratum

Meyerhof's method for estimating Qp in **Clay**

For piles in *saturated clays* under undrained conditions ($\phi = 0$), the net ultimate load can be given as

$$Q_p \approx N_c^* c_u A_p = 9 c_u A_p \qquad (9.18)$$

where c_u = undrained cohesion of the soil below the tip of the pile.

Table 9.5 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

Estimating Qp in SAND

Example 9.1

Consider a 20-m-long concrete pile with a cross section of 0.407 m × 0.407 m fully embedded in sand. For the sand, given: unit weight, $\gamma = 18$ kN/m^3; and soil friction angle, $\phi' = 35°$. Estimate the ultimate point Q_p with each of the following:

a. Meyerhof's method

Solution

Part a

From Eqs. (9.16) and (9.17),

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

For $\phi' = 35°$, the value of $N_q^* \approx 143$ (Table 9.5). Also, $q' = \gamma L = (18)(20) = 360$ kN/m^2. Thus,

$$A_p q' N_q^* = (0.407 \times 0.407)(360)(143) \approx 8528 \text{ kN}$$

Again,

$$A_p(0.5 p_a N_q^* \tan \phi') = (0.407 \times 0.407)[(0.5)(100)(143)(\tan 35)] \approx 829 \text{ kN}$$

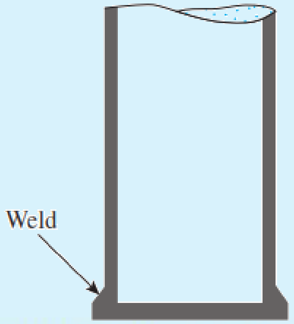
Hence, $Q_p = \mathbf{829\ kN}$.

Estimating Qp in CLAY

Example 9.2

Consider a pipe pile (flat driving point—see Figure 9.2d) having an outside diameter of 457 mm. The embedded length of the pile in layered saturated clay is 20 m. The following are the details of the subsoil:

Depth from ground surface (m)	Saturated unit weight, γ(kN/m³)	c_u(kN/m²)
0–3	16	25
3–10	17	40
10–30	18	90



(d)

The groundwater table is located at a depth of 3 m from the ground surface. Estimate Q_p by using

a. Meyerhof's method

Solution

Part a

From Eq. (9.18),

$$Q_p = 9c_uA_p$$

The tip of the pile is resting on a clay with $c_u = 90$ kN/m². So,

$$Q_p = (9)(90)\left[\left(\frac{\pi}{4}\right)\left(\frac{457}{1000}\right)^2\right] = \mathbf{132.9\ kN}$$

Estimating Qp in CLAY

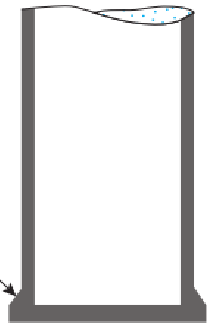
Example 9.2

Consider a pipe pile (flat driving point—see Figure 9.2d) having an outside diameter of 457 mm. The embedded length of the pile in layered saturated clay is 20 m. The following are the details of the subsoil:

Depth from ground surface (m)	Saturated unit weight, γ(kN/m^3)	c_u(kN/m^2)
0–3	16	25
3–10	17	40
10–30	18	90

The groundwater table is located at a depth of 3 m from the ground surface. Estimate Q_p by using

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



(d)

Solution

Part a

From Eq. (9.18),

$$Q_p = 9c_u A_p$$

The tip of the pile is resting on a clay with $c_u = 90$ kN/m^2. So,

$$Q_p = (9)(90)\left[\left(\frac{\pi}{4}\right)\left(\frac{457}{1000}\right)^2\right] = \mathbf{132.9\ kN}$$

Part b

From Eq. (9.31),

$$Q_p = A_p c_u N_c^*$$

From Eq. (9.35),

$$I_r = I_{rr} = 347\left(\frac{c_u}{p_a}\right) - 33 = 347\left(\frac{90}{100}\right) - 33 = 279.3$$

So use $I_{rr} = 279.3$.
From Table 9.8 for $I_{rr} = 279.3$, the value of $N_c^* \approx 11.4$. Thus,

$$Q_p = A_p c_u N_c^* = \left[\left(\frac{\pi}{4}\right)\left(\frac{457}{1000}\right)^2\right](90)(11.4) = \mathbf{168.3\ kN}$$

Note: The average value of Q_p is

$$\frac{132.9 + 168.3}{2} \approx \mathbf{151\ kN}$$

PROBLEM

Problems

9.1 A 20-m-long concrete pile is shown in Figure P9.1. Estimate the ultimate point load Q_p by

a. Meyerhof's method

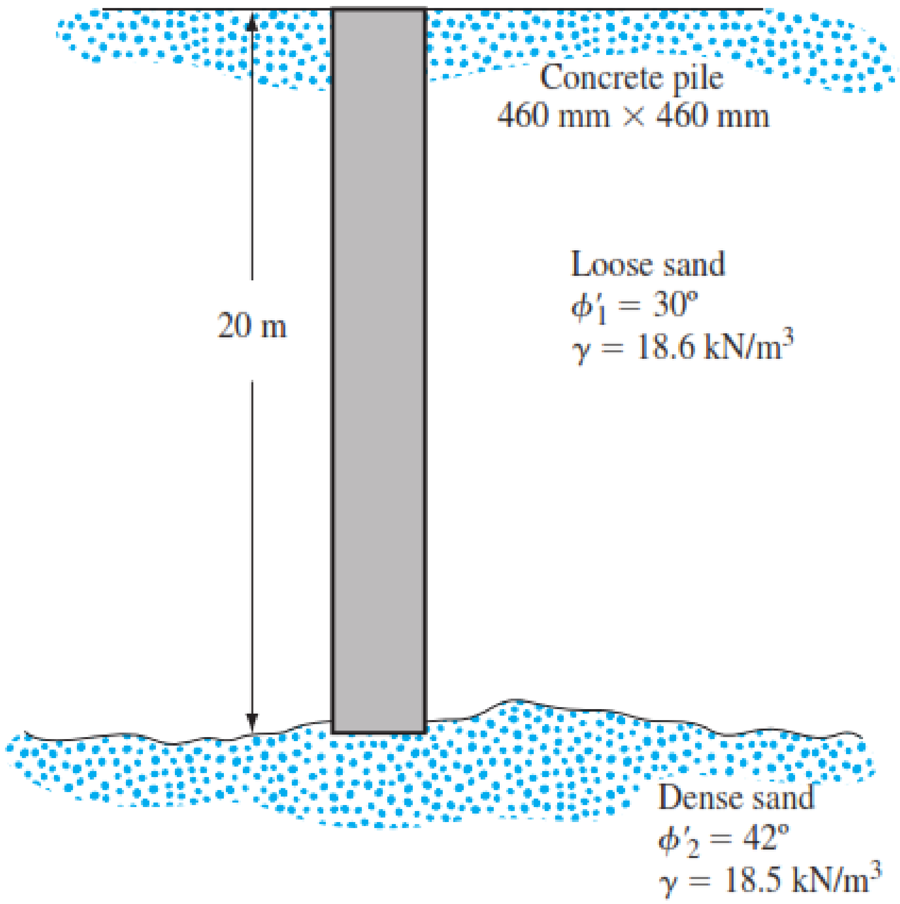


Figure P9.1

Estimating Friction Resistance, Qs

Estimating Friction Resistance (Qs) in SAND

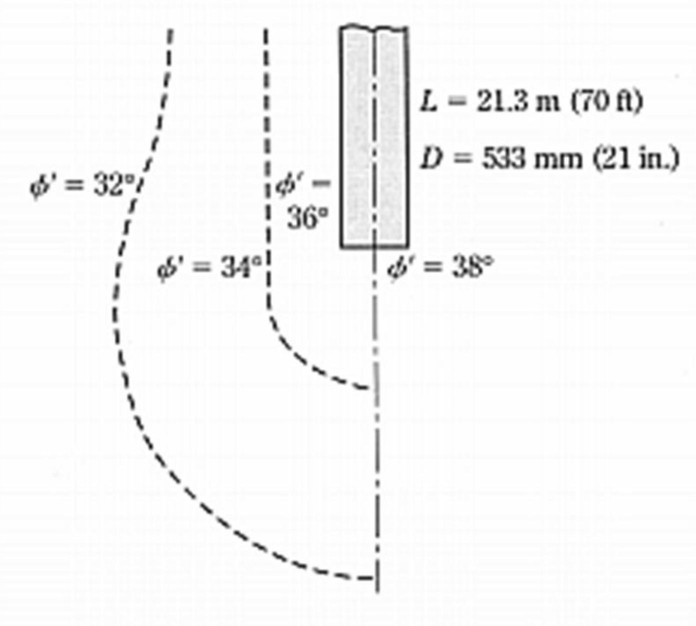
Friction Resistance in Sand

The frictional resistance (Qs):

$$Q_s = \Sigma p \, \Delta L f$$

The unit frictional resistance (*f*), is hard to estimate. In making an estimation of *f*, several important factor must be kept in mind:

1. The nature of the pile installation. For driven piles in sand, the vibration caused during pile driving helps density the soil around the pile. Figure shows the contours of the soil friction angle ϕ' around a driven pile.



Estimating Friction Resistance (Qs) in SAND

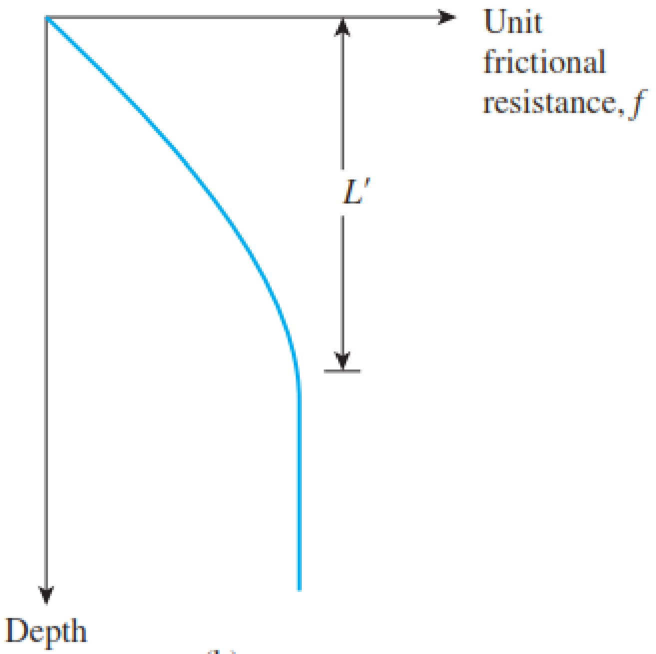
Friction Resistance in Sand

2. It has been observed that the nature of variation of f in field is approximately as shown in Figure. The unit skin friction increased with depth more or less linearly to a depth of L' and remains constant thereafter.

   L' = 15D.

3. At similar depths, the unit skin friction in loose sand is higher for a high- displacement pile, compared with a low-displacement pile.

4. At similar depths, bored, or jetted, piles will have a lower unit skin friction compared with **driven piles**.



(b)

Estimating Friction Resistance (Qs) in SAND

Friction Resistance in Sand

Taking into account the preceding factors, we can give the following approximate relationship for *f* (see Figure 9.16):

For z = 0 to L' $$f = K\sigma'_o \tan \delta'$$

For z = L' to L $$f = f_{z=L'}$$

In these equations,

K = effective earth pressure coefficient
σ'_o = effective vertical stress at the depth under consideration
δ' = soil-pile friction angle

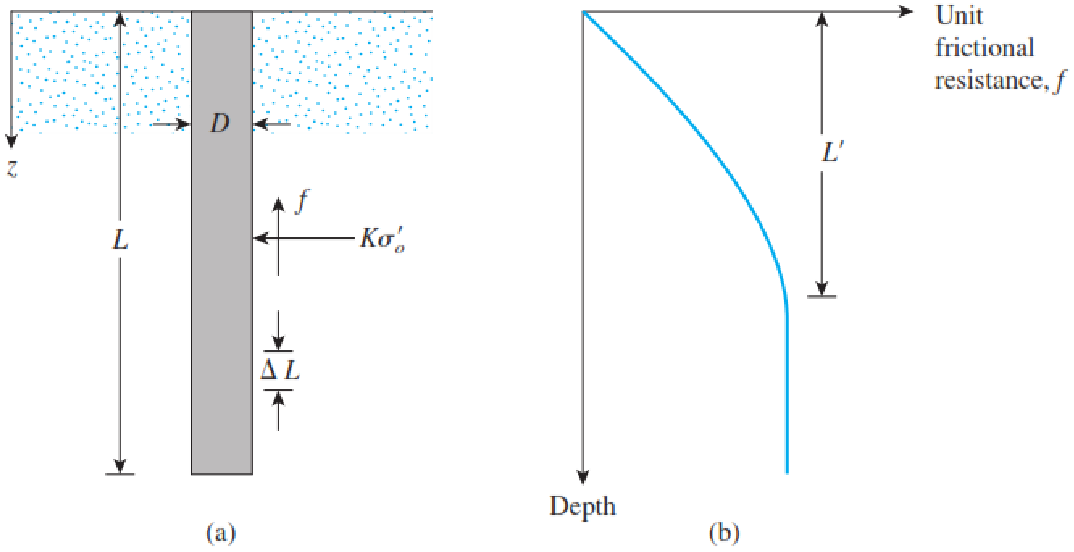


Figure 9.16 Unit frictional resistance for piles in sand

Estimating Friction Resistance (Qs) in SAND

In reality, the magnitude of K varies with depth; it is approximately equal to the Rankine passive earth pressure coefficient, K_p, at the top of the pile and may be less than the at-rest pressure coefficient, K_o, at a greater depth. Based on presently available results, the following average values of K are recommended for use in Eq. (9.41):

Pile type	K
Bored or jetted	$\approx K_o = 1 - \sin\phi'$
Low-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.4K_o = 1.4(1 - \sin\phi')$
High-displacement driven	$\approx K_o = 1 - \sin\phi'$ to $1.8K_o = 1.8(1 - \sin\phi')$

The values of δ' from various investigations appear to be in the range from $0.5\phi'$ to $0.8\phi'$.

Based on load test results in the field, Mansur and Hunter (1970) reported the following average values of K.

H-piles. . . . . . $K = 1.65$

Steel pipe piles. . . . . . $K = 1.26$

Precast concrete piles. . . . . . $K = 1.5$

$$f = K\sigma_o' \tan\delta'$$

Estimating Friction Resistance (Qs) in SAND

Berdasarkan API RP2A (1987)

$$f = K.\sigma'. \tan \delta$$

K= coefficient of lateral earth
= 0.8 (open ended piles)
= 1.0 (full displacement piles)
σ′= tegangan overburden efektif pada kedalaman yang ditinjau
δ= sudut friksi antara tanah dengan tiang

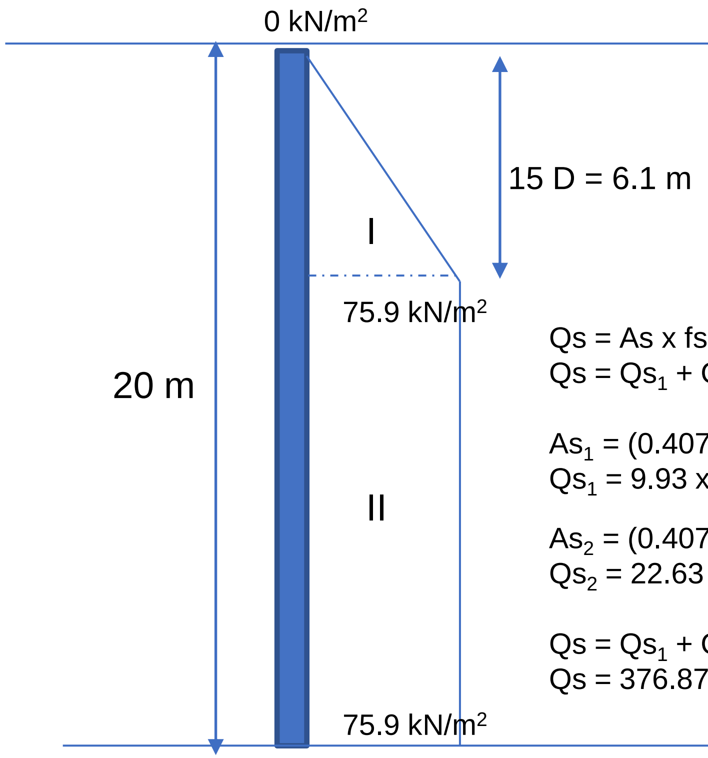
Soil	δ, degrees	Limiting f, kips/ft^2 (kPa)
very loose to medium, sand to silt	15	1.0 (47.8)
loose to dense, sand to silt	20	1.4 (67.0)
medium to dense, sand to sand-silt	25	1.7 (83.1)
dense to very dense, sand to sand-silt	30	2.0 (95.5)
dense to very dense, gravel to sand	35	2.4 (114.8)

Estimating Friction Resistance (Qs) in SAND

Example 9.1

Consider a 20-m-long concrete pile with a cross section of 0.407 m × 0.407 m fully embedded in sand. For the sand, given: unit weight, $\gamma = 18\ \text{kN/m}^3$; and soil friction angle, $\phi' = 35°$. Estimate the ultimate point Q_p with each of the following:

a. Meyerhof's method



Qs = As x fs

$Qs = Qs_1 + Qs_2$

$As_1 = (0.407 \times 4)\ 6.1 = 9.93\ m^2$

$Qs_1 = 9.93 \times 75.9 / 2 = 376.874$ kN

$As_2 = (0.407 \times 4)\ (20-6.1) = 22.63\ m^2$

$Qs_2 = 22.63 \times 75.9 = 1717.56$ Kn

$Qs = Qs_1 + Qs_2$

Qs = 376.874 + 1717.56 =2094.43 kN

Example 9.5

Refer to Example 9.1. For the pile, estimate the frictional resistance Q_s

Use $K = 1.3$ and $\delta' = 0.8\phi'$.

Consider a 20-m-long concrete pile with a cross section of 0.407 m × 0.407 m fully embedded in sand. For the sand, given: unit weight, $\gamma = 18\ \text{kN/m}^3$; and soil friction angle, $\phi' = 35°$. Estimate the ultimate point Q_p with each of the following:

Solution

Part a

From Eq. (9.40), $L' = 15D = (15)(0.407) \approx 6.1$ m. Refer to Eq. (9.41):

At $z = 0$: $\sigma'_o = 0$

$f = 0$

At $z = 6.1$ m: $\sigma'_o = (6.1)(18) = 109.8\ \text{kN/m}^2$

So

$$f = K\sigma'_o \tan\delta' = (1.3)(109.8)[\tan(0.8 \times 35)] \approx 75.9\ \text{kN/m}^2$$

Thus,

$$Q_s = \frac{(f_{z=0} + f_{z=6.1\text{m}})}{2} pL' + f_{z=6.1\text{m}}\, p(L - L')$$

$$= \left(\frac{0 + 75.9}{2}\right)(4 \times 0.407)(6.1) + (75.9)(4 \times 0.407)(20 - 6.1)$$

$$= 376.87 + 1717.56 = 2094.43\ \text{kN} \approx \mathbf{2094\ kN}$$

Daya dukung Ijin, Qall

Example 9.1

Example 9.5

$$Q_{all} = \frac{Q_p + Q_s}{F_S} = \frac{829\ kN + 2094\ kN}{3}$$

$$Q_{all} = 974.33\ kN$$

PROBLEM 1

Problems

9.1 A 20-m-long concrete pile is shown in Figure P9.1. Estimate the ultimate point load Q_p by

a. Meyerhof's method

9.2 Refer to the pile shown in Figure P9.1. Estimate the side resistance Q_s by

a. Using Eqs. (9.40) through (9.42). Use $K = 1.5$ and $\delta' = 0.6\phi'$

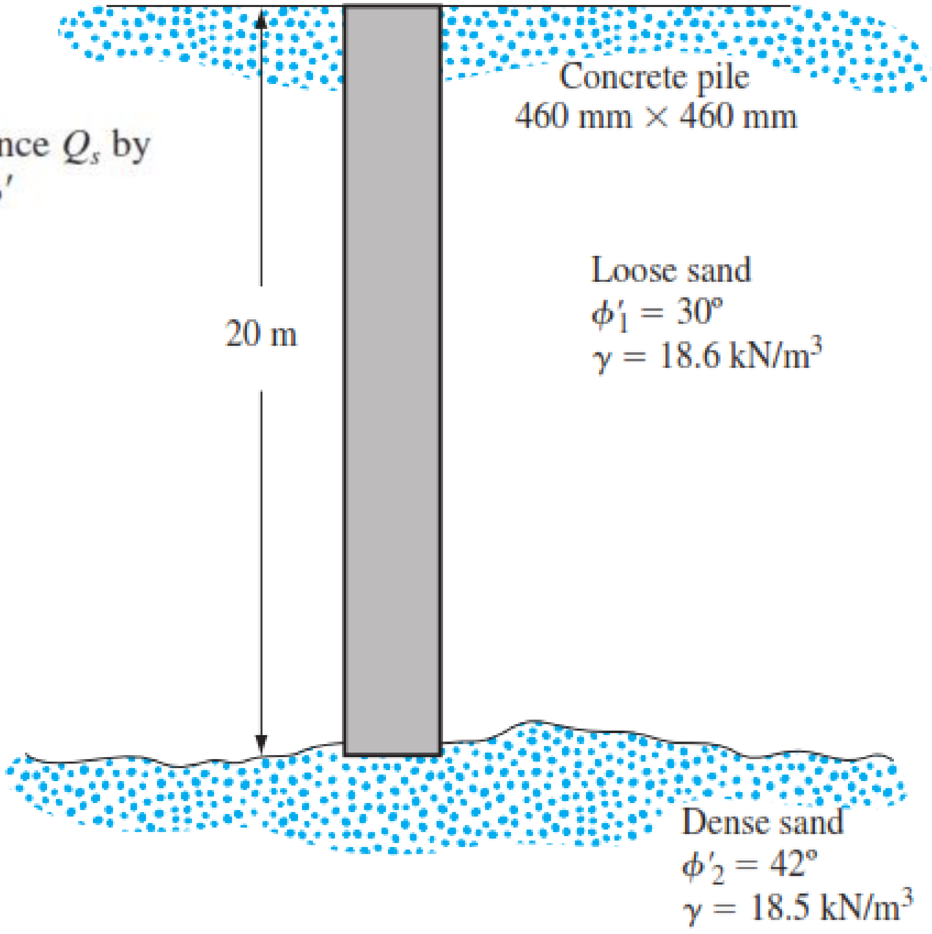


Figure P9.1

Estimating Friction Resistance (Qs) in CLAY

Friction Resistance in **Clay**

Estimating the frictional (or skin) resistance of piles in clay is almost as difficult a task as estimating that in sand, due to the presence of several variables that cannot easily be quantified. Several methods for obtaining the unit frictional resistance of piles are described in the literature:

1. λ Method
2. α Method
3. β Method

Estimating Friction Resistance (Qs) in CLAY

Frictional Resistance (Qs) in Clay λ Method

This method, proposed by Vijayvergiya and Focht (1972), is based on the assumption that the displacement of soil caused by pile driving results in a passive lateral pressure at any depth and that the average unit skin resistance is:

$$f_{av} = \lambda(\bar{\sigma}'_o + 2c_u)$$

where

$\bar{\sigma}'_o$ = mean effective vertical stress for the entire embedment length
c_u = mean undrained shear strength ($\phi = 0$)

The value of λ changes with the depth of penetration of the pile. The total frictional resistance may be calculated as:

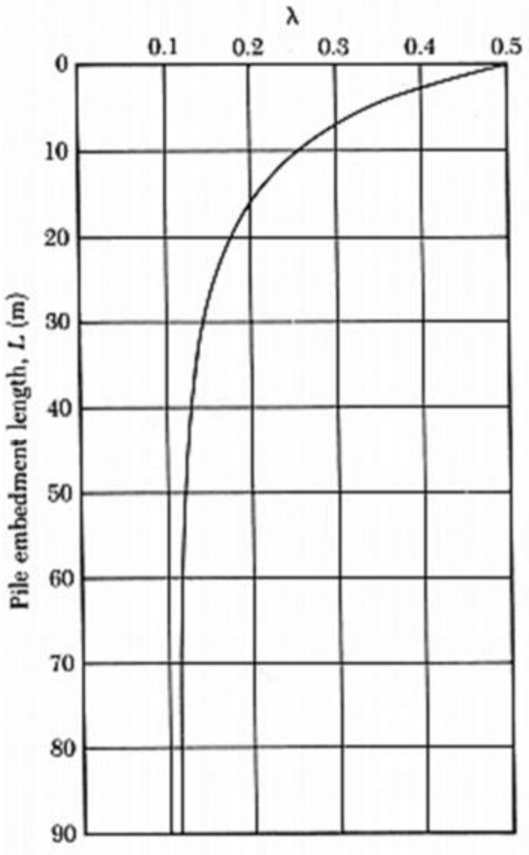
$$Q_s = pLf_{av}$$

Estimating Friction Resistance (Qs) in CLAY

Frictional Resistance (Qs) in Clay λ Method

Table 9.9 Variation of λ with Pile Embedment Length, L

Embedment length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110



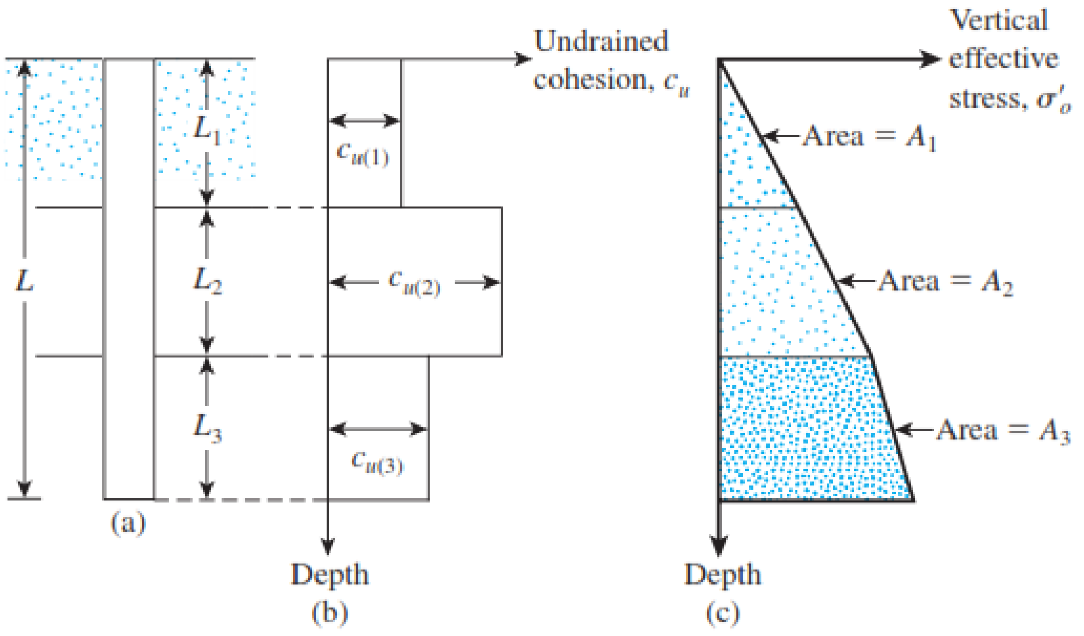


Figure 9.20 Application of λ method in layered soil

The mean effective stress :

$$\bar{\sigma}'_o = \frac{A_1 + A_2 + A_3 + \cdots}{L}$$

where $A_1, A_2, A_3, \ldots$ = areas of the vertical effective stress diagrams.

Estimating Friction Resistance (Qs) in CLAY

Frictional Resistance (Qs) in Clay α Method

The unit skin resistance in clayey soils can be represented by the equation:

$$f = \alpha c_u$$

where α = empirical adhesion factor

$$Q_s = \Sigma f p \, \Delta L = \Sigma \alpha c_u p \, \Delta L$$

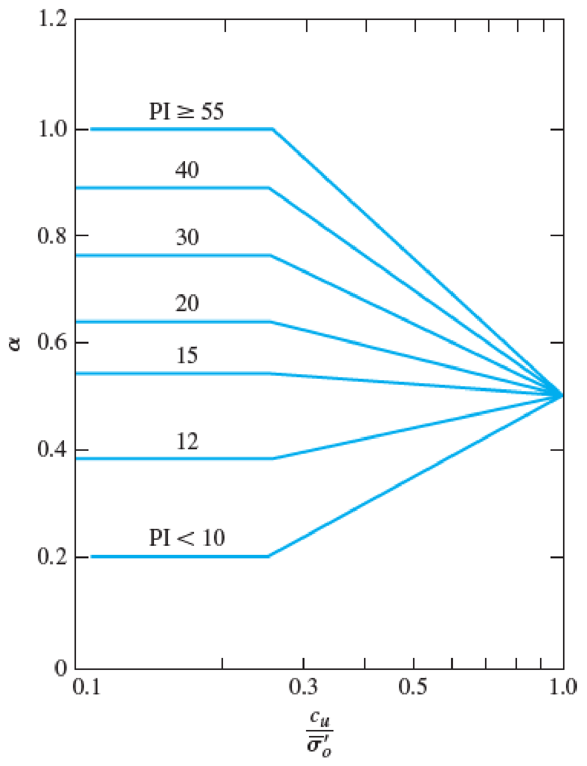


Figure 9.21 Variation of α with $c_u/\bar{\sigma}'_o$ for the NGI-99 method [Eqs. (9.57a) and (9.57b)]

Table 9.10 Variation of α (Interpolated Values Based on Terzaghi, Peck and Mesri, 1996)

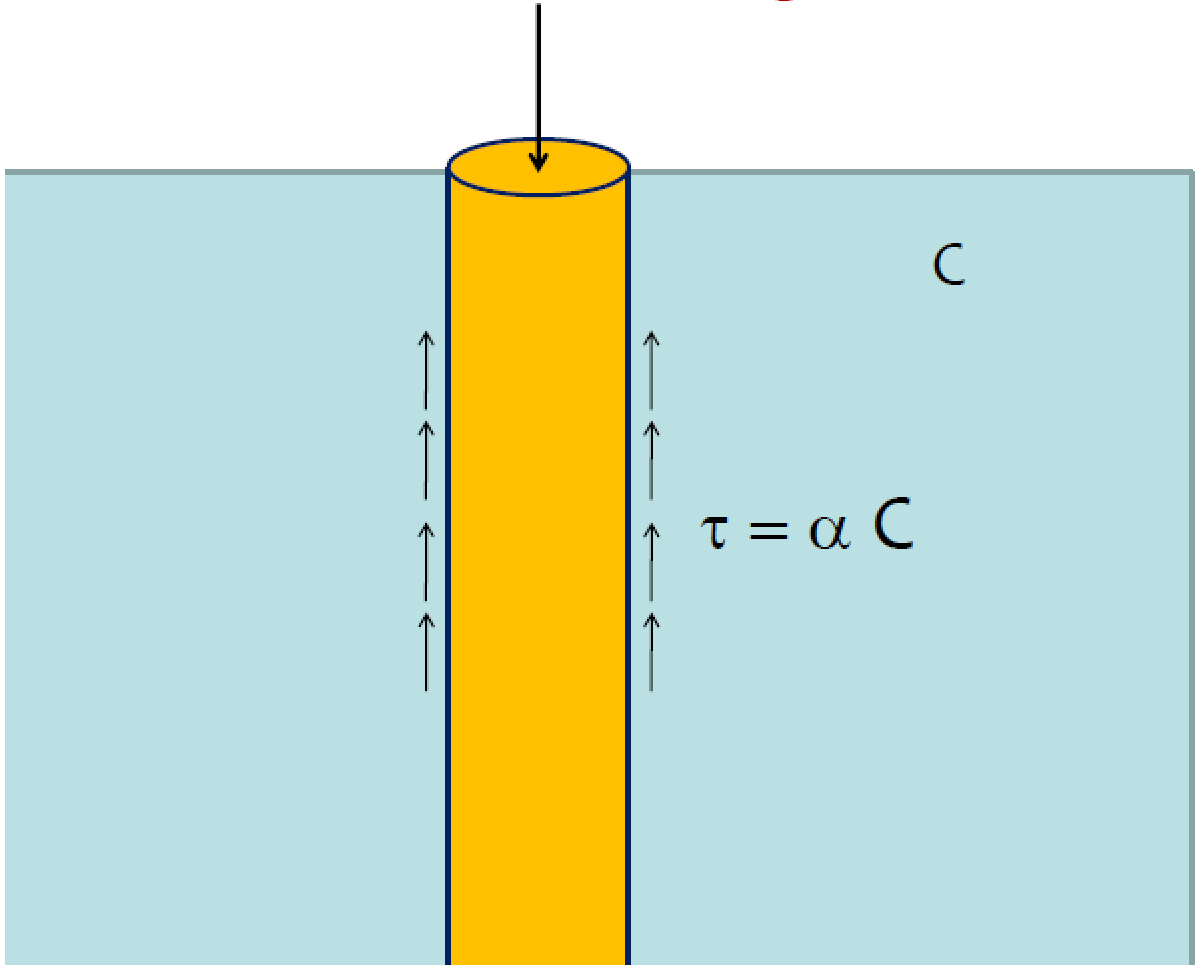
$\frac{c_u}{p_a}$	α
≤ 0.1	1.00
0.2	0.92
0.3	0.82
0.4	0.74
0.6	0.62
0.8	0.54
1.0	0.48
1.2	0.42
1.4	0.40
1.6	0.38
1.8	0.36
2.0	0.35
2.4	0.34
2.8	0.34

Note: p_a = atmospheric pressure ≈ 100 kN/m² or 2000 lb/ft²

Estimating Friction Resistance (Qs) in CLAY

Frictional Resistance (Qs) in Clay α Method

Tahanan Geser Selimut Tiang Pada Tanah Lempung :



Estimating Friction Resistance (Qs) in CLAY

Frictional Resistance (Qs) in Clay β Method

When piles are driven into saturated clays, the pore water pressure in the soil around the piles increases. The excess pore water pressure in normally consolidated clays may be four to six times *cu* . However, within a month or so, this pressure gradually dissipates. Hence, the unit frictional resistance for the pile can be determined on the basis of the effective stress parameters of the clay in a remolded state (*c*’ = 0). Thus, at any depth,

$$\boxed{f = \beta\sigma'_o}$$

$$Q_s = \Sigma fp\,\Delta L$$

where

σ'_o = vertical effective stress
$\beta = K \tan \phi'_R$
ϕ'_R = drained friction angle of remolded clay
K = earth pressure coefficient

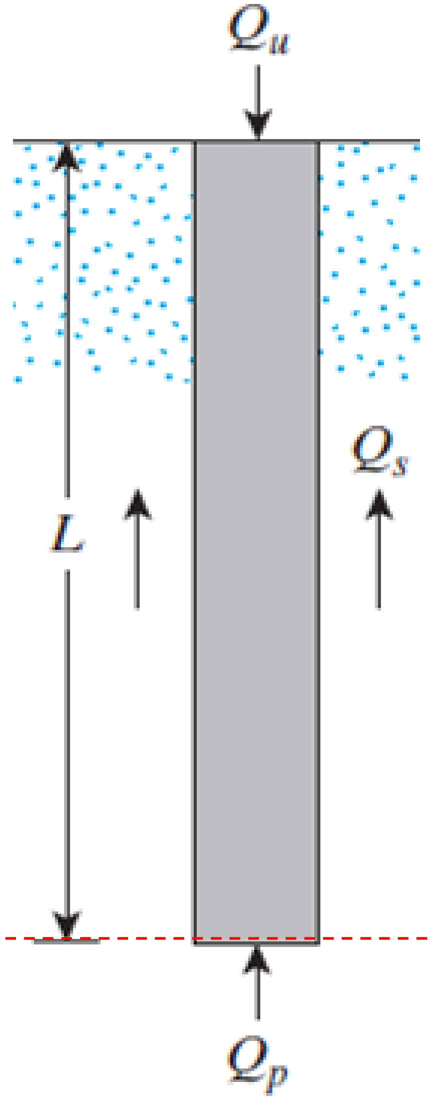
Conservatively, the magnitude of K is the earth pressure coefficient at rest, or

$$K = 1 - \sin \phi'_R \qquad \text{(for normally consolidated clays)}$$

$$K = (1 - \sin \phi'_R)\sqrt{\text{OCR}} \qquad \text{(for overconsolidated clays)}$$

where OCR = overconsolidation ratio.

Resume daya dukung aksial tiang tunggal



Sand	Clay
$Q_s = \Sigma p \, \Delta L f$ L' = 15D. δ' = 2/3 to 4/5 ϕ' For z = 0 to L' $f = K\sigma'_o \tan \delta'$ For z = L' to L $f = f_{z=L'}$	$Q_s = \Sigma p \, \Delta L f$ $f_{av} = \lambda(\overline{\sigma}'_o + 2c_u)$ λ Method $f = \alpha c_u$ α Method $f = \beta \sigma'_o$ β Method
$Q_p = A_p q' N_q^* \leq A_p q_l$ $q_l = 0.5 \, p_a N_q^* \tan \phi'$	$Q_p \approx N_c^* c_u A_p = 9 c_u A_p$

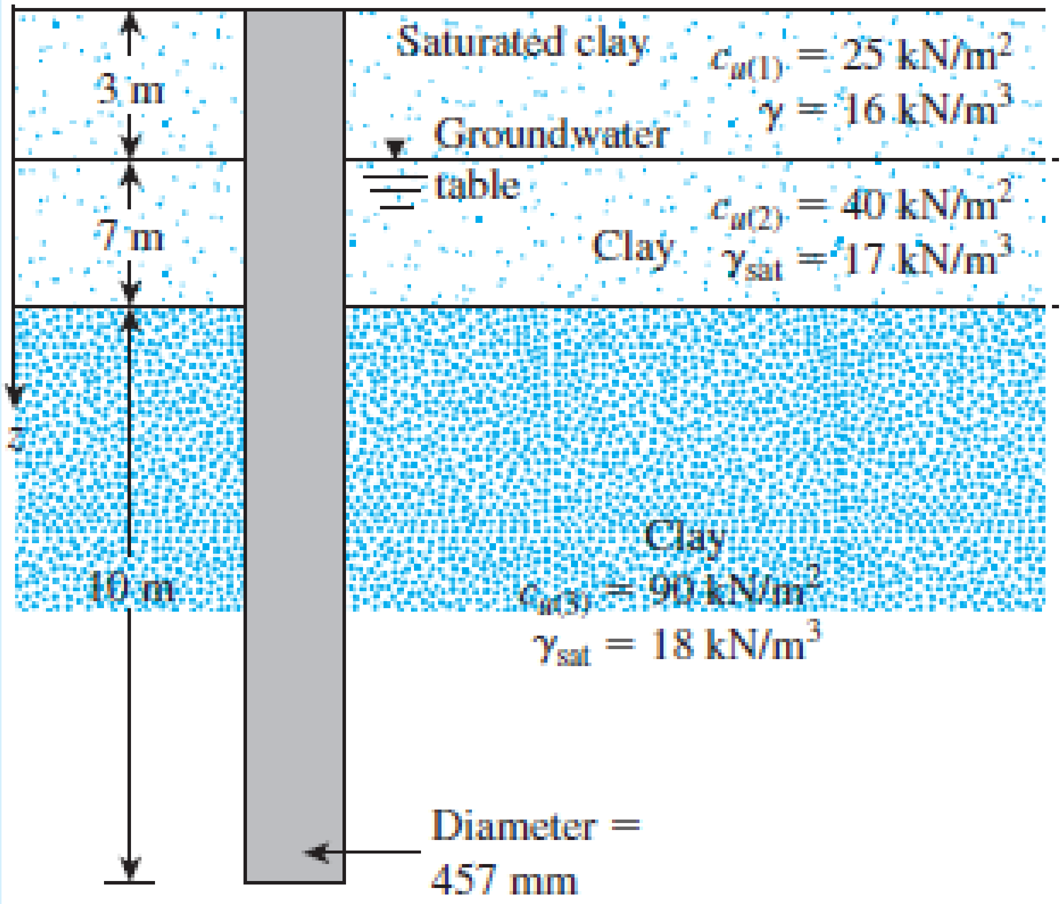
EXAMPLE

Example 9.7

Refer to the pipe pile in saturated clay shown in Figure 9.24. For the pile,

a. Calculate the skin resistance (Q_s) by (1) the α method, (2) the λ method, and (3) the β method. For the β method, use $\phi'_R = 30°$ for all clay layers. The top 10 m of clay is normally consolidated. The bottom clay layer has an OCR = 2. (Note: diameter of pile = 457 mm)

b. Using the results of Example 9.2, estimate the allowable pile capacity (Q_{all}). Use FS = 4.



EXAMPLE (solution)

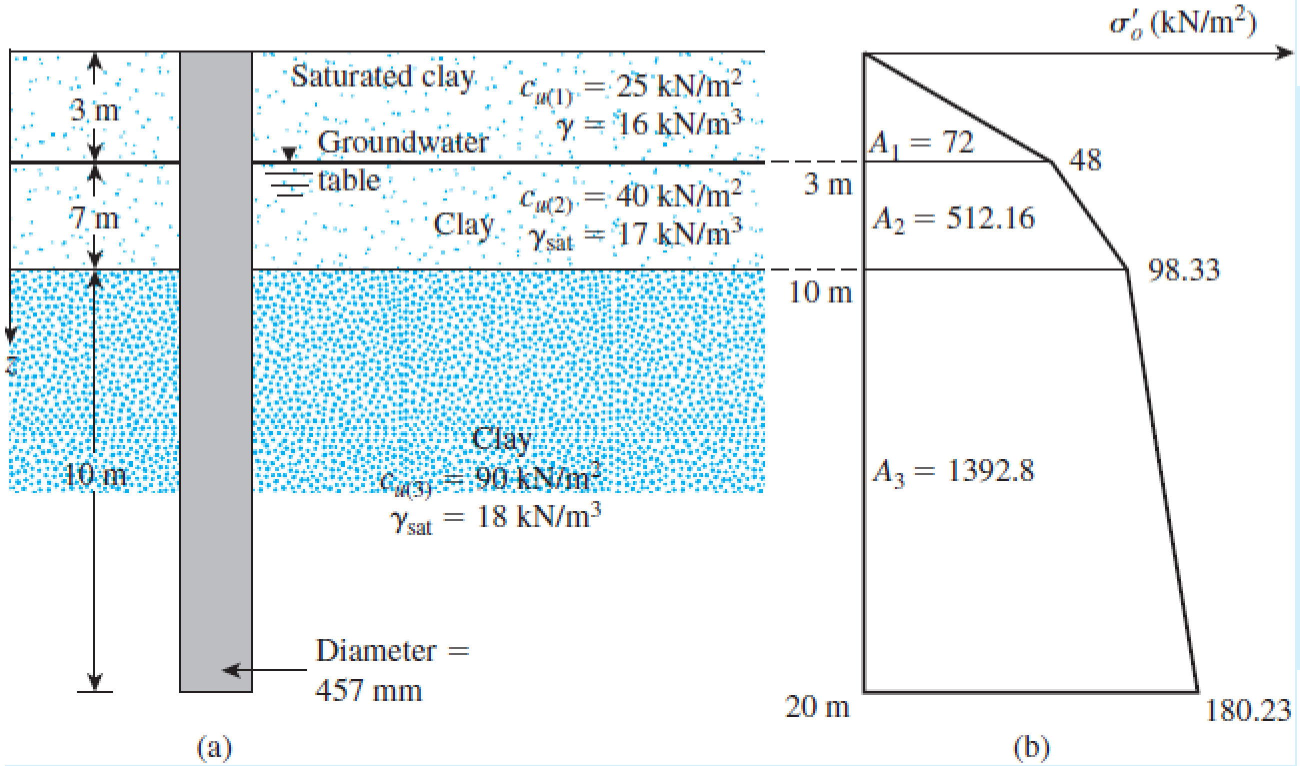


Figure 9.24 Estimation of the load bearing capacity of a driven-pipe pile

Part a

(1) From Eq. (9.59),

$$Q_s = \Sigma \alpha c_u p \Delta L$$

[*Note:* $p = \pi(0.457) = 1.436$ m] Now the following table can be prepared.

Depth (m)	**ΔL (m)**	**c_u (kN/m^2)**	**α (Table 9.10)**	**$\alpha c_u p \Delta L$ (kN)**
0–3	3	25	0.87	93.7
3–10	7	40	0.74	297.5
10–20	10	90	0.51	659.1
				$Q_s \approx 1050$ kN

EXAMPLE (solution)

(2) From Eq. 9.51, $f_{av} = \lambda(\overline{\sigma}'_o + 2c_u)$. Now, the average value of c_u is

$$\frac{c_{u(1)}(3) + c_{u(2)}(7) + c_{u(3)}(10)}{20} = \frac{(25)(3) + (40)(7) + (90)(10)}{20} = 62.75 \text{ kN/m}^2$$

To obtain the average value of $\overline{\sigma}'_o$, the diagram for vertical effective stress variation with depth is plotted in Figure 9.24b. From Eq. (9.52),

$$\overline{\sigma}'_o = \frac{A_1 + A_2 + A_3}{L} = \frac{72 + 512.16 + 1392.8}{20} = 98.85 \text{ kN/m}^2$$

From Table 9.9, the magnitude of λ is 0.173. So

$$f_{av} = 0.173[98.85 + (2)(62.75)] = 38.81 \text{ kN/m}^2$$

Hence,

$$Q_s = pLf_{av} = \pi(0.457)(20)(38.81) = \mathbf{1114.4\ kN}$$

EXAMPLE (solution)

(3) The top layer of clay (10 m) is normally consolidated, and $\phi'_R = 30°$. For $z = 0–3$ m, from Eq. (9.64), we have

$$f_{av(1)} = (1 - \sin \phi'_R) \tan \phi'_R \, \overline{\sigma}'_o$$
$$= (1 - \sin 30°)(\tan 30°)\left(\frac{0 + 48}{2}\right) = 6.93 \text{ kN/m}^2$$

Similarly, for $z = 3–10$ m.

$$f_{av(2)} = (1 - \sin 30°)(\tan 30°)\left(\frac{48 + 98.33}{2}\right) = 21.12 \text{ kN/m}^2$$

For $z = 10–20$ m from Eq. (9.65),

$$f_{av} = (1 - \sin \phi'_R)\tan \phi'_R \sqrt{\text{OCR}} \, \overline{\sigma}'_o$$

For OCR = 2,

$$f_{av(3)} = (1 - \sin 30°)(\tan 30°)\sqrt{2}\left(\frac{98.33 + 180.23}{2}\right) = 56.86 \text{ kN/m}^2$$

EXAMPLE (solution)

So,

$$Q_s = p[f_{av(1)}(3) + f_{av(2)}(7) + f_{av(3)}(10)]$$
$$= (\pi)(0.457)[(6.93)(3) + (21.12)(7) + (56.86)(10)] = \mathbf{1058.45\ kN}$$

Part b

$$Q_u = Q_p + Q_s$$

From Example 9.2,

$$Q_p \approx 151\ \text{kN}$$

Again, the values of Q_s from the α method, λ method, and β method are close. So,

$$Q_s = \frac{1050 + 1114.4 + 1058.45}{3} \approx 1074\ \text{kN}$$

$$Q_{all} = \frac{Q_u}{FS} = \frac{151 + 1074}{4} = 306.25\ \text{kN} \approx \mathbf{306\ kN}$$

PROBLEM 2 (Home work)

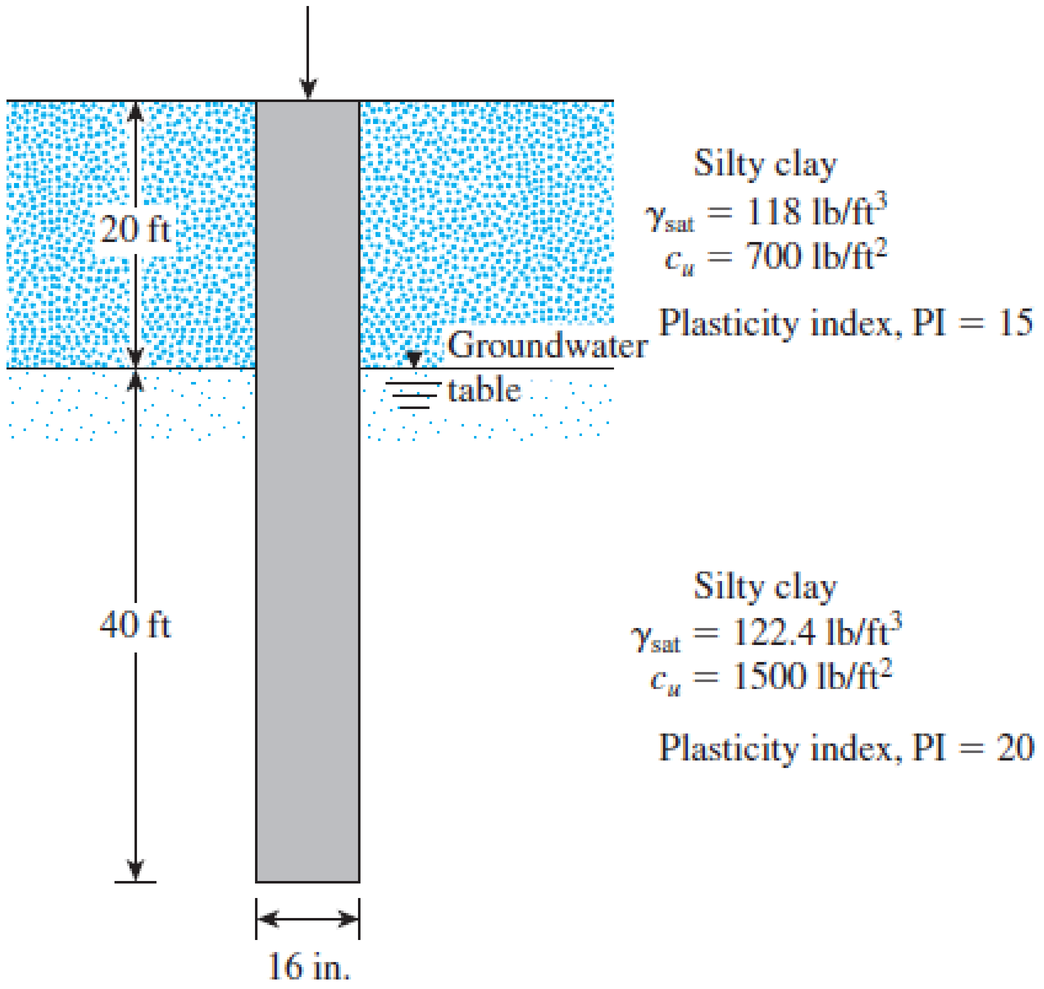
9.10 A concrete pile 16 in. × 16 in. in cross section is shown in Figure P9.10. Calculate the ultimate skin friction resistance by using the

a. α method [use Eq. (9.59) and Table 9.10]

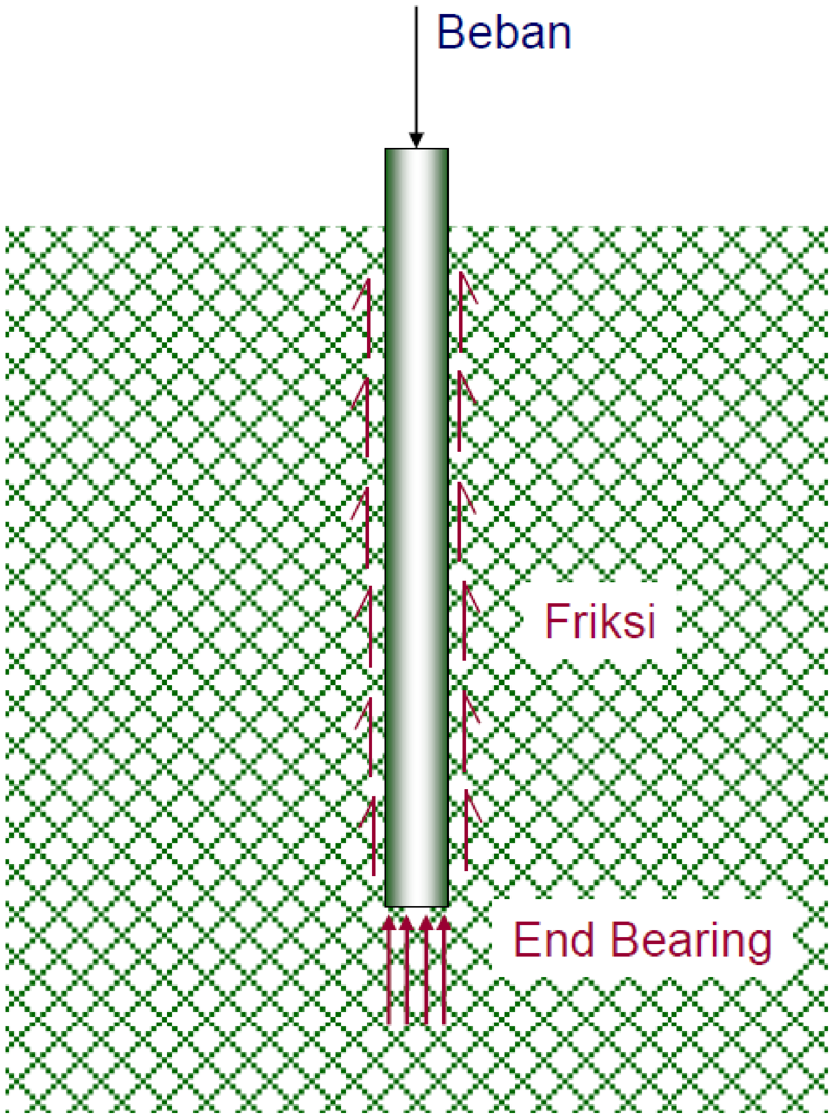
b. λ method

c. β method

Use $\phi'_R = 20°$ for all clays, which are normally consolidated.



Nspt Parameter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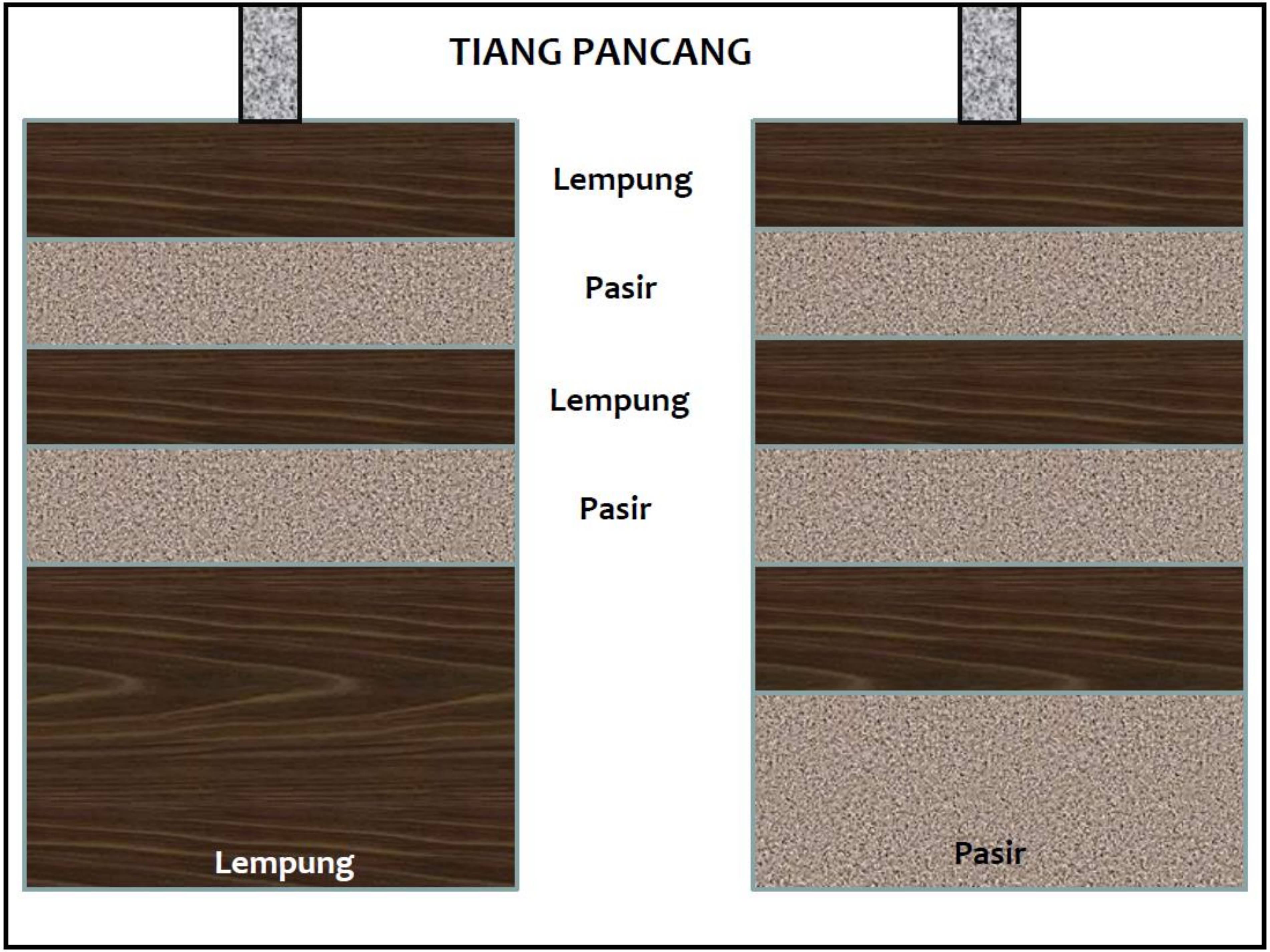
End Bearing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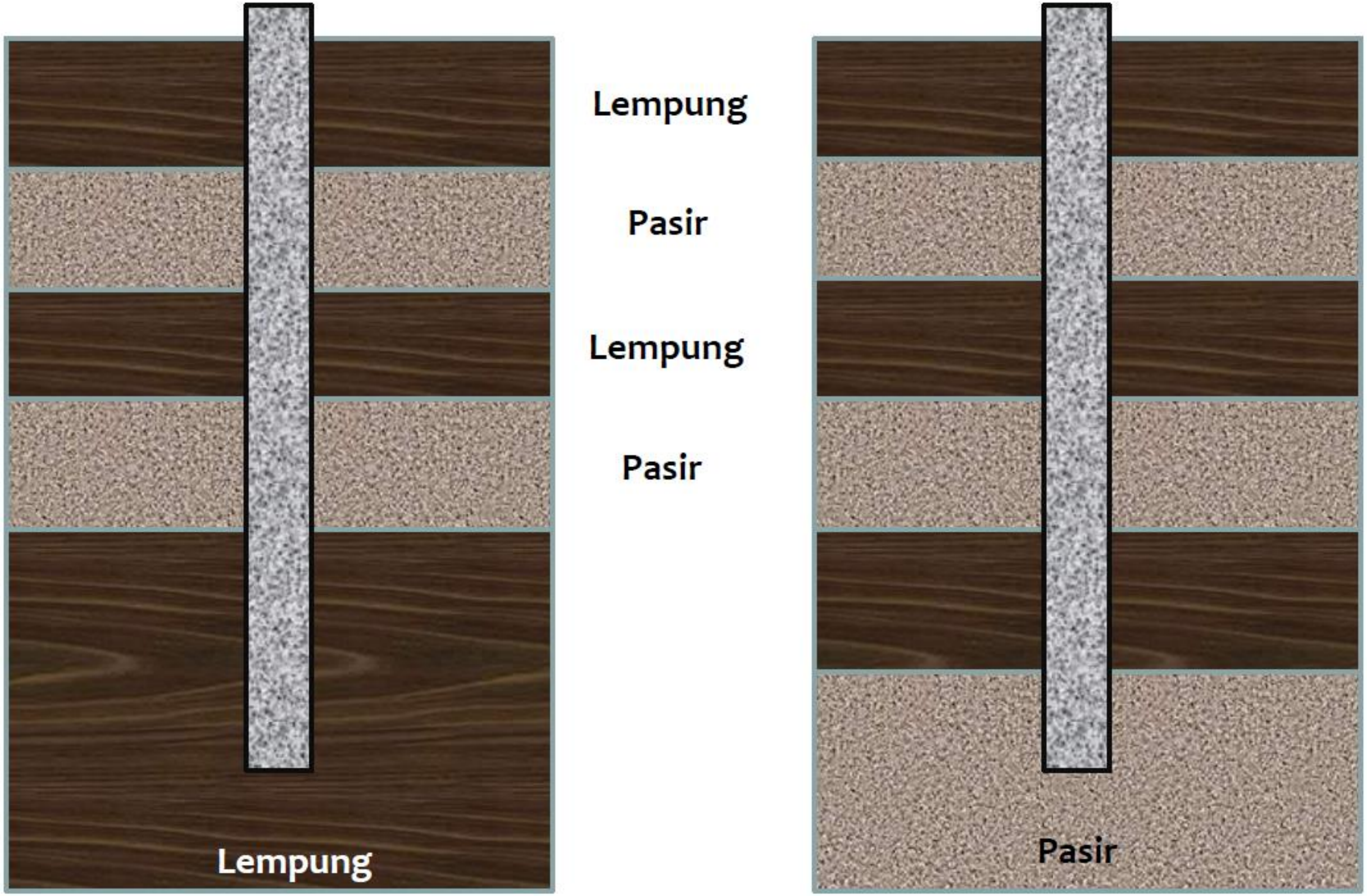
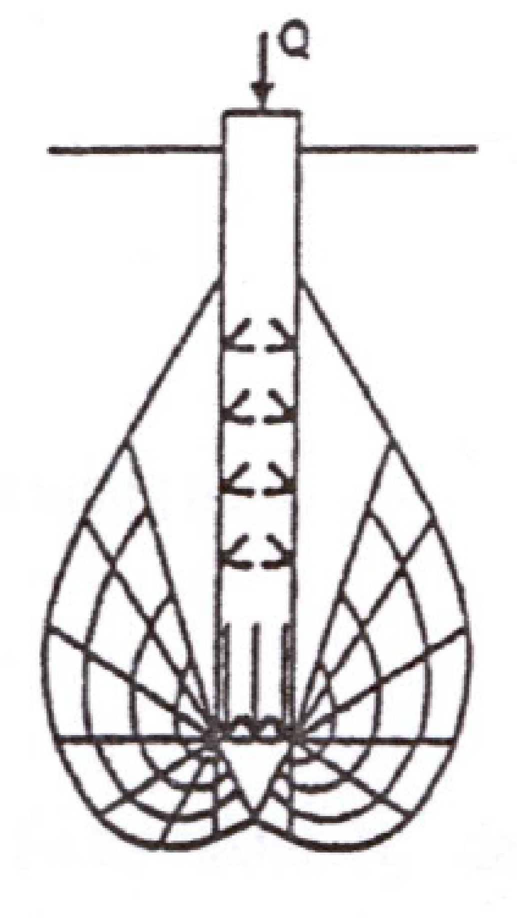
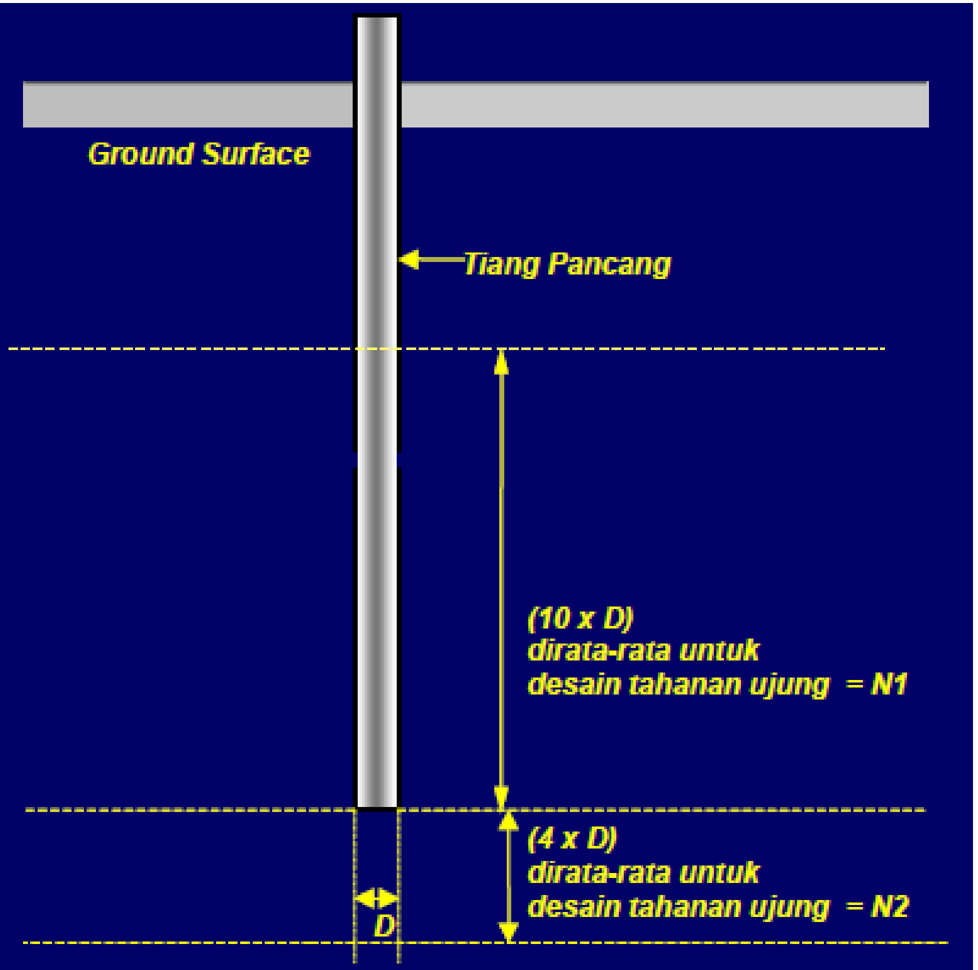


Nspt Parameter

Estimating Point Bearing Capacity, Qp

Daya Dukung Ujung Tiang Pancang untuk tanah Pasir





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$$N_{desain} = \frac{1}{2}\,(N1 + N2)$$

Daya Dukung Ujung Tiang Pancang untuk tanah Pasir

Daya Dukung Ujung untuk Tanah Pasiran dng Korelasi Empiris N-SPT

Meyerhoff Theory

→Tiang Pancang :

$$Q_p = 40 \times N\ SPT \times A_p \quad \text{(unit dlm ton)}$$

Dimana,

N–SPT = (N1+N2)/2

N1= harga rata-rata N dari dasar ke 10-D keatas

N2= harga rata-rata N dari dasar ke 4-D kebawah

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Daya Dukung Ujung Tiang Bor untuk tanah Pasir

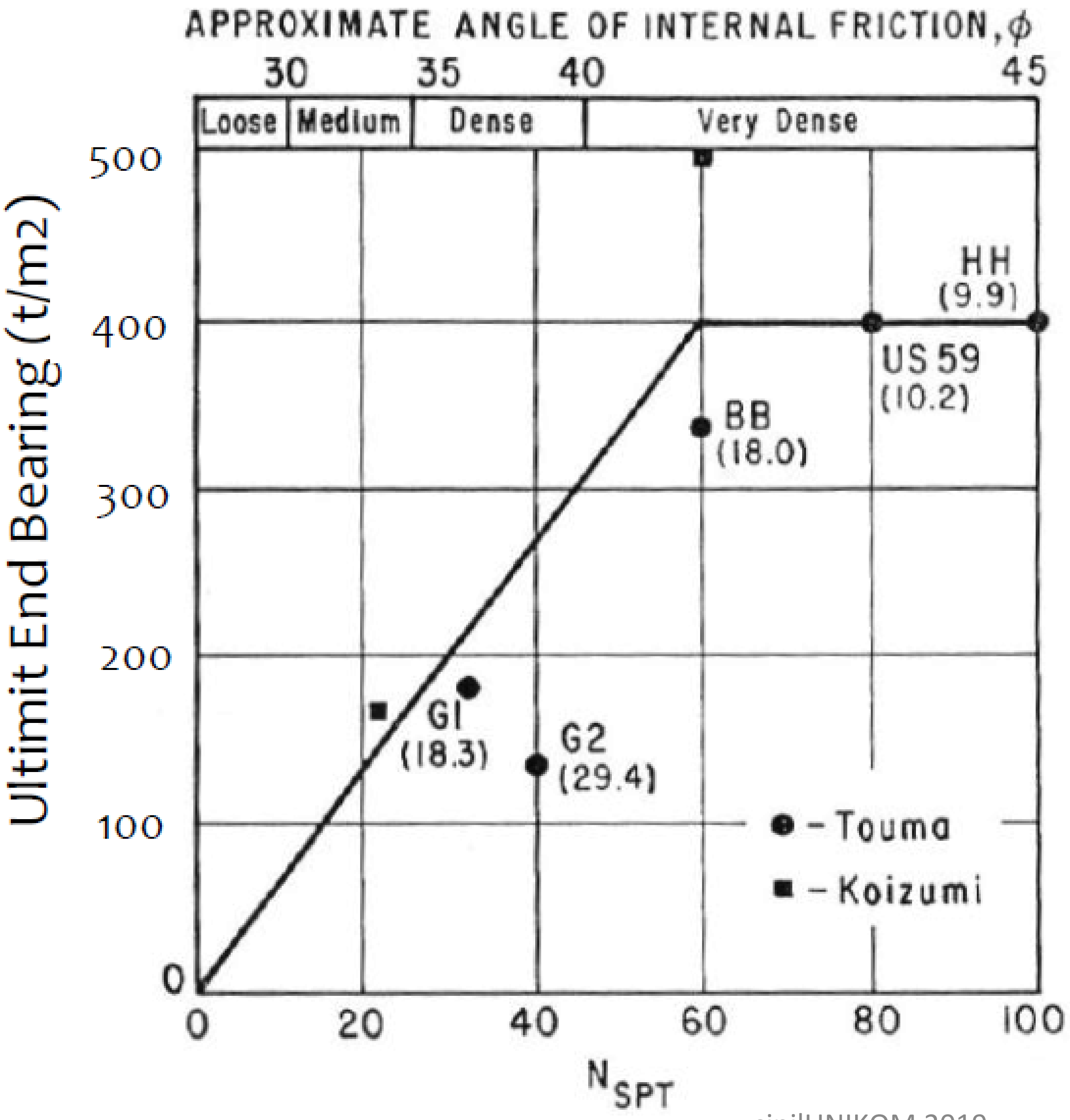
Table 4.4 Summary of procedures for estimating base resistance (q_p) of drilled shafts in sand

REFERENCE	DESCRIPTION
Touma and Reese (1974)	Loose q_p (tsf) = 0; Medium Dense q_p (tsf) = $\frac{16}{k}$; Very Dense q_p (tsf) = $\frac{40}{k}$; { k = 1 for D_p < 1.67 ft & k = 0.6D_p for $D_p \geq$ 1.67 ft.
Meyerhof (1976)	q_p (tsf) = $\frac{2N_{corr}D_b}{15D_p}$ < $\frac{4}{3}N_{corr}$ for sand; < N_{corr} for nonplastic silts
Quiros and Reese (1977)	Same as Touma and Reese (1974)
Reese and Wright (1977)	q_p (tsf) = $\frac{2}{3}$ N =7 N (t/m2) for N ≤ 60; q_p (tsf) = 40 =400 (t/m2) for N > 60
Reese and O'Neill (1988)	q_p (tsf) = 0.6N for N ≤ 75; q_p (tsf) = 45 for N > 75

where N_{corr} = SPT blow count corrected for overburden pressure
= $[0.77\log_{10}(20/\sigma_v')]N$
N = uncorrected SPT blow count
D_p = base diameter of drilled shaft in ft
D_b = embedment of drilled shaft in sand bearing layer

Qp = 7 N (t/m^2) < 400 (t/m^2)

Daya Dukung Ujung Tiang Bor untuk tanah Pasir



Qp = 7 N (t/m²) < 400 (t/m²)

Daya Dukung Ujung Tiang Bor dan Tiang Pancang untuk tanah Lempung

Daya Dukung Ujung untuk Tanah Kohesif C_u

$$\boxed{q_u = cN_c + \cancel{qN_q} + \cancel{\tfrac{1}{2}\gamma BN_\gamma}}$$

Tiang Pancang dan Tiang Bor:

$$Qp = Nc \times C_u \times Ap$$
$$= 9 \times C_u \times Ap$$

Nspt Parameter

Estimating Friction Capacity, Qs

Correlation with Standard Penetration Test Results

Meyerhof (1976) indicated that the average unit frictional resistance, f_{av}, for high-displacement driven piles may be obtained from average standard penetration resistance values as

$$f_{av} = 0.02p_a(\overline{N}_{60}) \tag{9.45}$$

where

$(\overline{N}_{60})$ = average value of standard penetration resistance
p_a = atmospheric pressure ($\approx$100 kN/m^2 or 2000 lb/ft^2)

For low-displacement driven piles

$$f_{av} = 0.01p_a(\overline{N}_{60}) \tag{9.46}$$

Daya Dukung Friksi Tiang Pancang dan Tiang Bor untuk tanah Pasir

Tahanan Geser Selimut Tiang dari Tanah Berpasir Dari Korelasi Empiris dng N-SPT
(Menurut Naval Engineering Facilities Command)

1. Tiang Pancang :

$Qs = 0.2 \times (N\ SPT)_{N60} \times Li \times p$ (ton) displasemen besar, tiang tertutup

$Qs = 0.1 \times (N\ SPT)_{N60} \times Li \times p$ (ton) displasemen kecil, tiang terbuka

2. Tiang Bor :

$Qs = 0.1 \times (N\ SPT)_{N60} \times Li \times p$ (ton)

Handout Masyur Iryam, 2016

N_{60}

Faktor koreksi N_{SPT} yaitu $(N_1)_{60}$, $(N_1)_{60}$ adalah kondisi terkoreksi 1 atm dan 60%energi hammer

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S$$

N_{SPT}

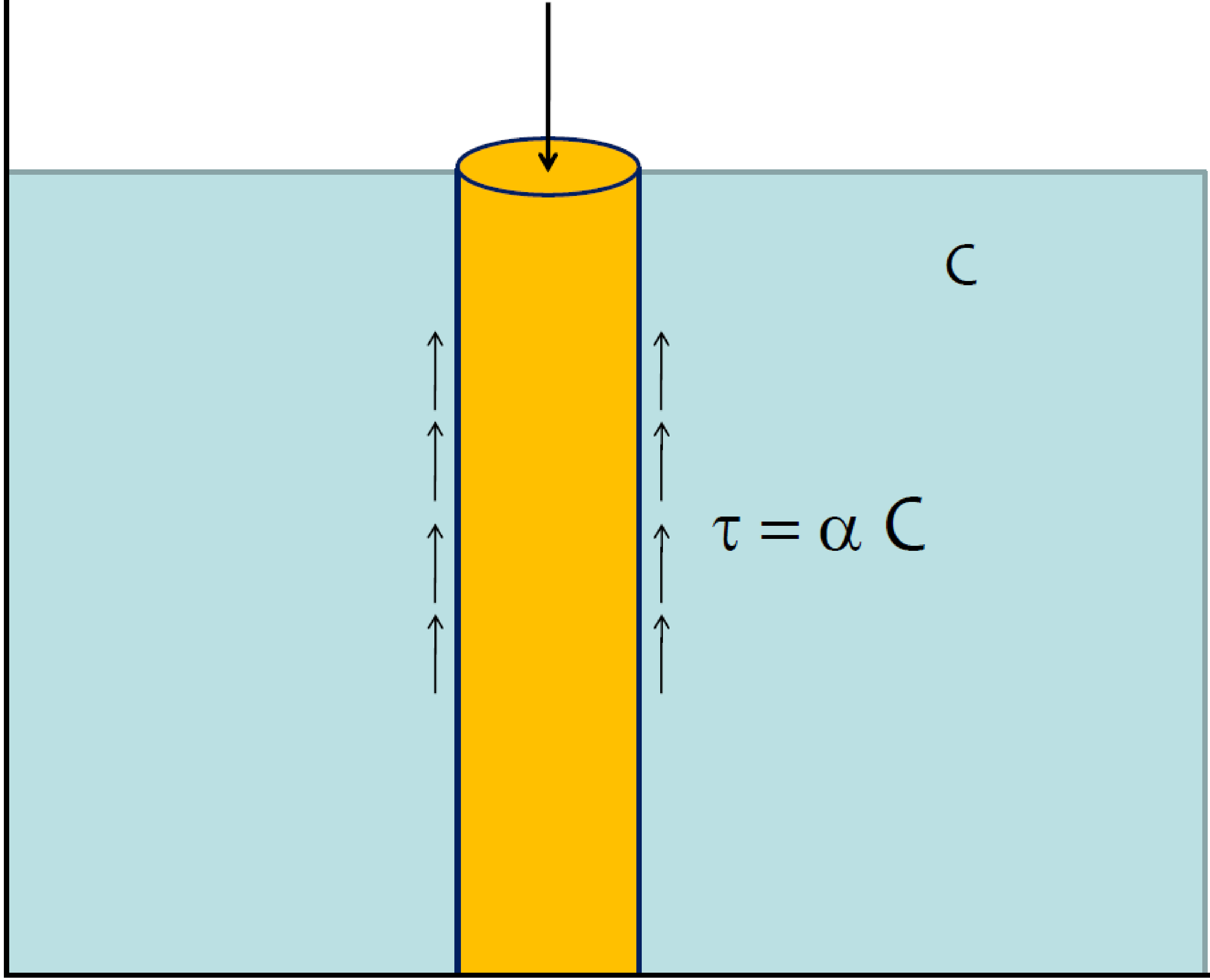
Nilai C_N Maximum = 1.7

$$C_N = 2.2/(1.2 + \sigma'_{vo}/P_a)$$

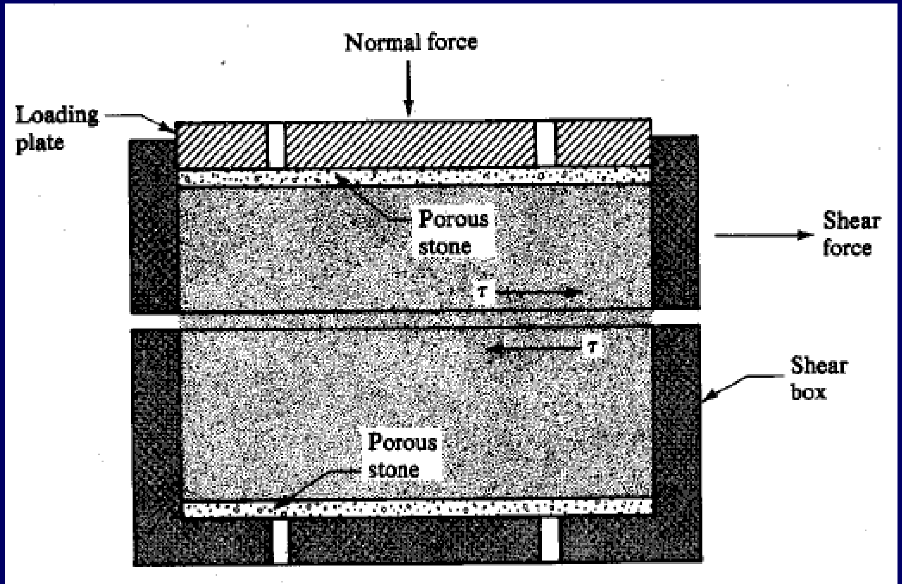
TABLE 2. Corrections to SPT (Modified from Skempton 1986) as Listed by Robertson and Wride (1998)

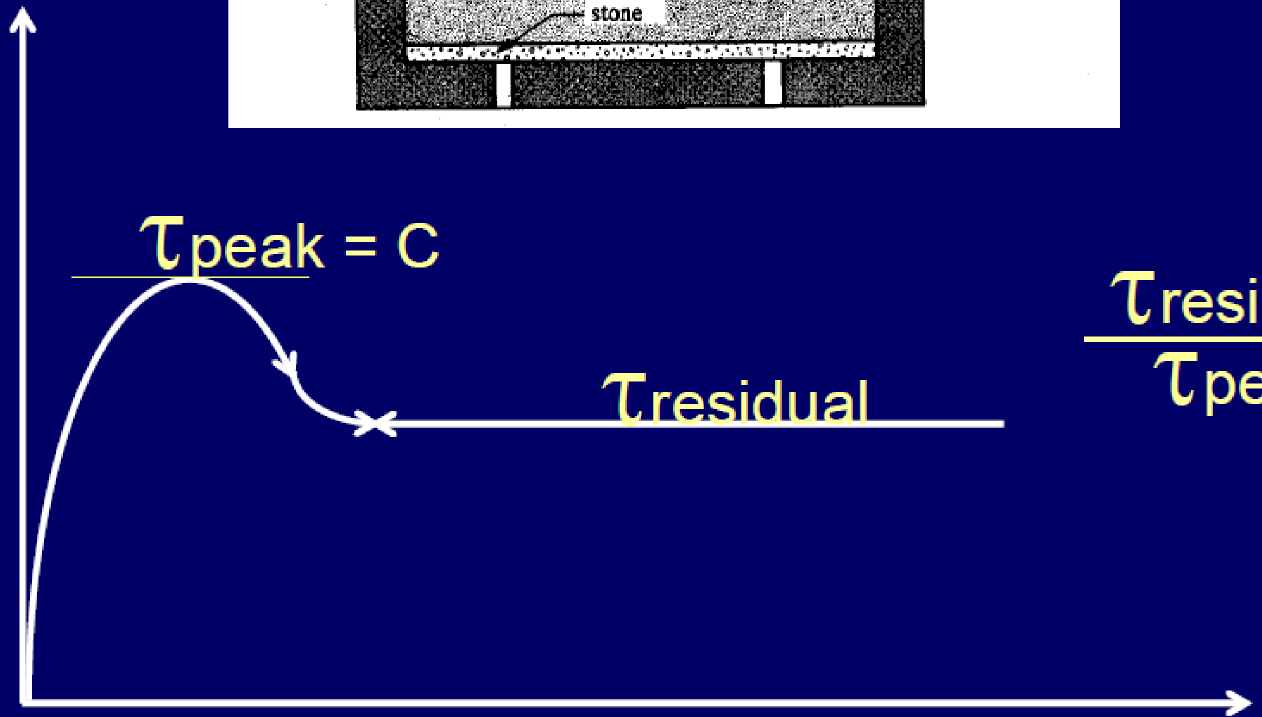
Factor	Equipment variable	Term	Correction
Overburden pressure	—	C_N	$(P_a/\sigma'_{vo})^{0.5}$
Overburden pressure	—	C_N	$C_N \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5–1.0
Energy ratio	Safety hammer	C_E	0.7–1.2
Energy ratio	Automatic-trip Donut-type hammer	C_E	0.8–1.3
Borehole diameter	65–115 mm	C_B	1.0
Borehole diameter	150 mm	C_B	1.05
Borehole diameter	200 mm	C_B	1.15
Rod length	<3 m	C_R	0.75
Rod length	3–4 m	C_R	0.8
Rod length	4–6 m	C_R	0.85
Rod length	6–10 m	C_R	0.95
Rod length	10–30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
Sampling method	Sampler without liners	C_S	1.1–1.3

Tahanan Geser Selimut Tiang Pada Tanah Lempung :



Apa arti α untuk tiang pancang?

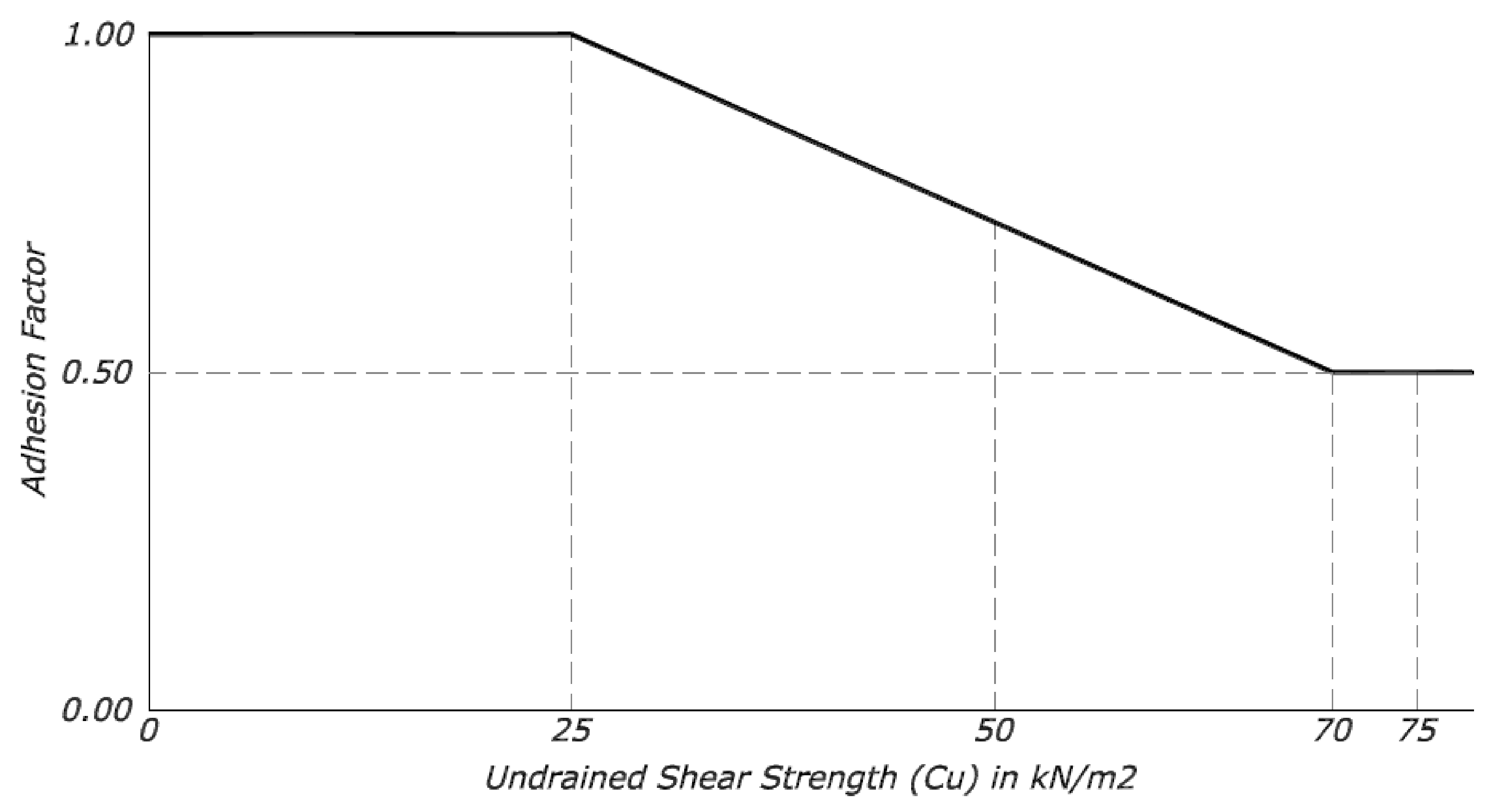




$$\frac{\tau_{residual}}{\tau_{peak}} = \alpha$$

Faktor Adhesi (α) pada Tanah Kohesif "Tiang Pancang" :

1. API Metode-2, 1986

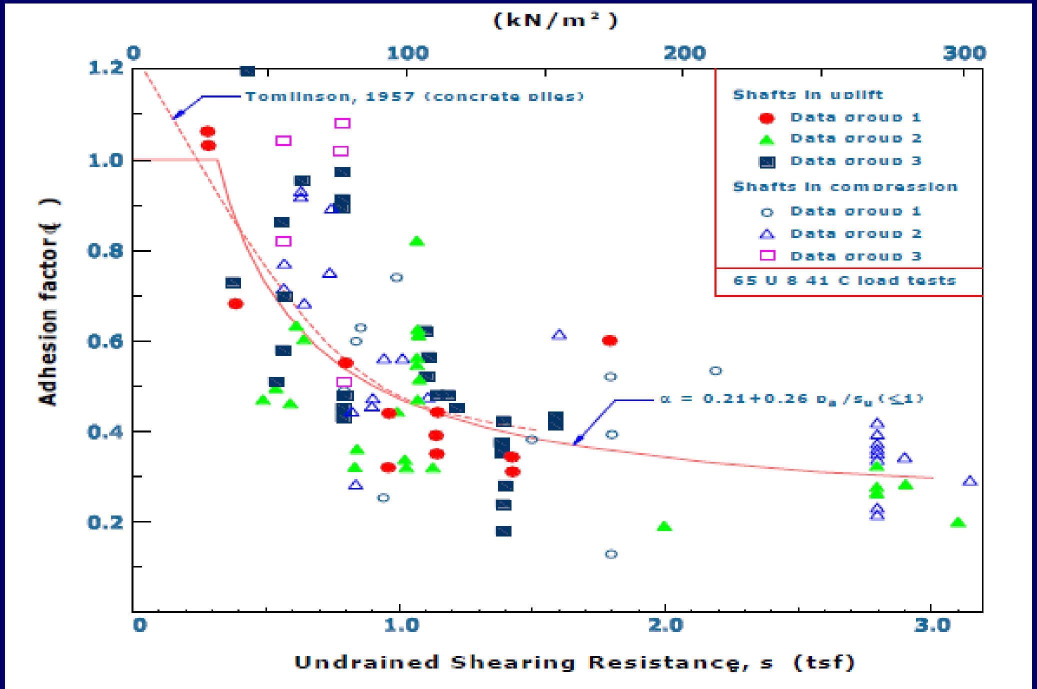


Faktor Adhesi (α) pada Tanah Kohesif "Tiang Bor" :

1. Reese and Wright, 1977 :

Menurut Reese dan Wright koefisien α untuk bored pile adalah 0.55

2. Kulhawy, 1984



Tahanan Geser Selimut Tiang Pada Tanah Lempung :

1. *Tahanan gerse selimut tiang yang merupakan kontribusi dari Cohesi Tanah adalah:*

$$Qs = (\alpha\, Cu)\ Li.\ P$$

α = *Koefisien adhesi antara tanah dan tiang*

Cu = *Undrained Cohesion*

Li = *Panjang lapisan tanah*

p = *keliling tiang*

Tahanan Geser Selimut Tiang dari Tanah Berpasir Dari Korelasi Empiris dng N-SPT (Menurut Naval Engineering Facilities Command)

1. Tiang Pancang :

   Qs = 0.2 x (N SPT) x Li x p (ton) displasemen besar, tiang tertutup

   Qs = 0.1 x (N SPT) x Li x p (ton) displasemen kecil, tiang terbuka

2. Tiang Bor :

   Qs = 0.1 x (N SPT) x Li x p (ton)

Rangkuman Perhitungan Daya Dukung Aksial Pondasi Tiang

Lempung		Pasir	
Tiang Pancang	**Tiang Bor**	**Tiang Pancang**	**Tiang Bor**
<u>Berdasarkan c</u> $\tau = \alpha c$ – α (API, 1986); Gbr. 3.15 – α (revised API meth, 1987) $\alpha = 0.5(\psi)^{-0.5}$, untuk $y \leq 1.0$ $\alpha = 0.5(\psi)^{-0.25}$, untuk $y > 1.0$ – α (Tomlinson, 1977); Gbr 3.16	<u>Berdasarkan c</u> $\tau = \alpha c$ – α (Kulhawy, 1984); Gbr. 3.17 – α = 0.55 (Reese & Wright, 1988) – α (Reese & O'Neil,1988); Gbr 3.18	<u>Dasar N-SPT</u> $\tau = 0.1N$ (t/m^2); (displacement kecil) $\tau = 0.2N$ (t/m^2); (displacement besar) <u>Dasar API RP2A 1987</u> $\tau = K\sigma' \tan\delta$ K = 0.8 (open ended piles) K = 1.0 (full displacement piles) $\delta \rightarrow$ Tbl. 3.5	<u>Dasar N-SPT</u> $\tau = (0.1\text{–}0.32)N$ (t/m^2) $\tau_{rata\text{-}rata} = 0.2N$ (t/m^2) (Rata-rata antara Meyerhof-1976 dan Reese & Wright-1977)
$q_p = 9c$		<u>Dasar N-SPT (Meyerhoff)</u> $q_p = 40N$ $(t/m^2) < 1600$ (t/m^2) N= (N1+N2)/2 <u>Dasar ϕ (API, 1986)</u> $q_p = \sigma' N_q$ $N_q \rightarrow$ Tbl. 3.2	<u>Dasar N-SPT (Reese & Wright,1977)</u> $q_p = 7N$ $(t/m^2) < 400$ (t/m^2) N= (N1+N2)/2 <u>Dasar N-SPT NAVDOC</u> $q_p = 13N$ (t/m^2) (1/3 tiang pancang) N = (N1+N2)/2

$$P_{ult} = 2\pi r\, \Sigma(\Delta L\, \tau) + \pi r^2\, q_p$$

Daya Dukung Friksi Tiang Pancang dan Tiang Bor untuk tanah Pasir

Depth	Sample Type	Graphic Log	USCS Symbol	Thickness (m)	Soil Description	Standard Penetration Test N1	N2	N3	N-SPT	N-SPT Curve (5 15 25 35 45 55 / 0 10 20 30 40 50 60)
0.0										
0.5										
1.0										
1.5										
2.0			MH		clayey SILT, grey, medium	1/15	3/15	2/15	5	
2.5										
3.0										
3.5										
4.0						2/15	5/15	7/15	12	
4.5										
5.0										
5.5			SM		silty SAND, black, medium					
6.0						3/15	4/15	4/15	8	
6.5										
7.0										
7.5										
8.0			SW		fine SAND, black, medium	9/15	1 1/15	7/15	18	
8.5										
9.0										
9.5										
10.0						1 1/15	8/15	7/15	15	
10.5										
11.0										
11.5										
12.0			ML		sandy SILT, grey, medium	1 3/15	6/15	9/15	15	
12.5										
13.0										
13.5										
14.0						7/15	9/15	8/15	17	
14.5										
15.0										

Daya Dukung Friksi Tiang Pancang dan Tiang Bor untuk tanah Pasir

GWL = 20 m
ϕ_R = 20 degrees
D tiang = 16 in circle
15D = 240 m
FS= 2.5

D = 0.5 m
Beban = 17 lantai (1 lantai asumsi 2 ton/m2)
Asumsi beban single pile = 340 kN/m^2

10 D = 5
4 D = 2

340 -1
340 -15

Depth	Soil Type	γ'	σ_o'	N_{spt}	N_{spt} Factor					$(N_1)_{60}$	Q_P			Q_s		Q_{ult}	Q_{all}
m		kN/m^3	kN/m^2		C_N	C_E	C_B	C_R	C_S		N desain	QP (ton)	Qp (kN)	fs (kN/m^2)	Qs (kN)	kN	kN
1	S	15.5	15.5	0	1.623616	0.8	1.05	0.8	1	0.0000	2.448	19.224	192.241	0.000	0.000	192.241	76.896
2	S	15.5	31	5	1.456954	0.8	1.05	0.8	1	4.8954	4.911	38.574	385.737	9.791	15.379	401.116	160.446
3	S	15.5	46.5	5	1.466471	0.8	1.05	0.8	1	4.9273	7.353	57.752	577.518	9.855	30.859	608.377	243.351
4	S	17	68	12	1.212678	0.8	1.05	0.8	1	9.7790	9.263	72.750	727.501	19.558	61.581	789.082	315.633
5	S	17	85	12	1.084652	0.8	1.05	0.8	1	8.7466	7.161	56.240	562.396	17.493	89.059	651.455	260.582
6	S	15.5	93	8	1.036952	0.8	1.05	0.8	1	5.5747	5.368	42.159	421.593	11.149	106.572	528.165	211.266
7	S	15.5	108.5	8	0.960031	0.8	1.05	0.8	1	5.1611	7.767	60.999	609.994	10.322	122.787	732.780	293.112
8	S	17	136	18	0.857493	0.8	1.05	0.8	1	10.3722	10.076	79.134	791.339	20.744	155.372	946.710	378.684
9	S	17	153	18	0.808452	0.8	1.05	0.8	1	9.7790	8.755	68.762	687.618	19.558	186.094	873.711	349.485
10	S	17	170	15	0.766965	0.8	1.05	0.8	1	7.7310	7.551	59.306	593.063	15.462	210.381	803.445	321.378
11	S	17	187	15	0.731272	0.8	1.05	0.8	1	7.3712	7.214	56.661	566.611	14.742	233.539	800.150	320.060
12	S	17	204	15	0.70014	0.8	1.05	0.8	1	7.0574	6.919	54.342	543.415	14.115	255.710	799.125	319.650
13	S	17	221	15	0.672673	0.8	1.05	0.8	1	6.7805	7.093	55.707	557.068	13.561	277.012	834.080	333.632
14	S	17	238	17	0.648204	0.8	1.05	0.8	1	7.4051	7.280	57.173	571.733	14.810	300.276	872.009	348.804
15	S	17	255	17	0.626224	0.8	1.05	0.8	1	7.1540	7.154	56.187	561.873	14.308	322.751	884.623	353.849

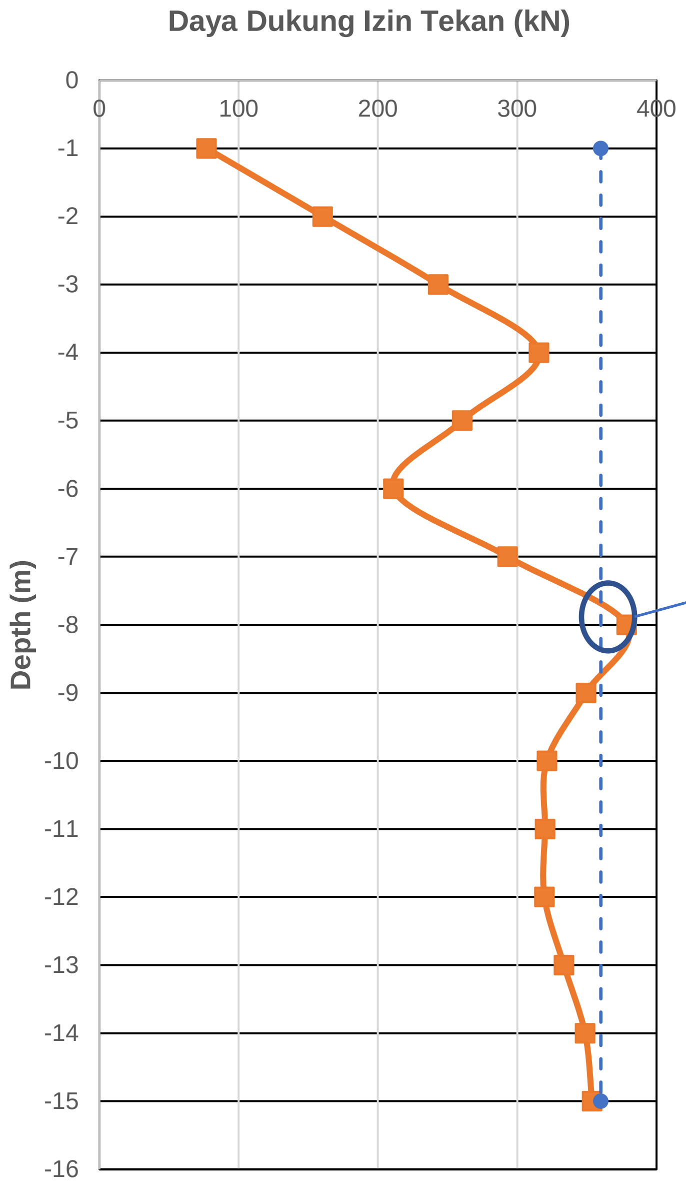
Daya Dukung Friksi Tiang Pancang dan Tiang Bor untuk tanah Pasir

N–SPT = (N1+N2)/2
N1= harga rata-rata N dari dasar ke 10-D keatas
N2= harga rata-rata N dari dasar ke 4-D kebawah

Korelasi Nspt dan γ

Depth	Soil Type	γ'	σ_o'	N_{spt}	N_{spt} Factor					$(N_1)_{60}$	Q_P			Q_s		Q_{ult}	Q_{all}
m		kN/m^3	kN/m^2		C_N	C_E	C_B	C_R	C_S		N desain	QP (ton)	Qp (kN)	fs (kN/m^2)	Qs (kN)	kN	kN
1	S	15.5	15.5	0	1.623616	0.8	1.05	0.8	1	0.0000	2.448	19.224	192.241	0.000	0.000	192.241	76.896
2	S	15.5	31	5	1.456954	0.8	1.05	0.8	1	4.8954	4.911	38.574	385.737	9.791	15.379	401.116	160.446
3	S	15.5	46.5	5	1.466471	0.8	1.05	0.8	1	4.9273	7.353	57.752	577.518	9.855	30.859	608.377	243.351
4	S	17	68	12	1.212678	0.8	1.05	0.8	1	9.7790	9.263	72.750	727.501	19.558	61.581	789.082	315.633
5	S	17	85	12	1.084652	0.8	1.05	0.8	1	8.7466	7.161	56.240	562.396	17.493	89.059	651.455	260.582
6	S	15.5	93	8	1.036952	0.8	1.05	0.8	1	5.5747	5.368	42.159	421.593	11.149	106.572	528.165	211.266
7	S	15.5	108.5	8	0.960031	0.8	1.05	0.8	1	5.1611	7.767	60.999	609.994	10.322	122.787	732.780	293.112
8	S	17	136	18	0.857493	0.8	1.05	0.8	1	10.3722	10.076	79.134	791.339	20.744	155.372	946.710	378.684
9	S	17	153	18	0.808452	0.8	1.05	0.8	1	9.7790	8.755	68.762	687.618	19.558	186.094	873.711	349.485
10	S	17	170	15	0.766965	0.8	1.05	0.8	1	7.7310	7.551	59.306	593.063	15.462	210.381	803.445	321.378
11	S	17	187	15	0.731272	0.8	1.05	0.8	1	7.3712	7.214	56.661	566.611	14.742	233.539	800.150	320.060
12	S	17	204	15	0.70014	0.8	1.05	0.8	1	7.0574	6.919	54.342	543.415	14.115	255.710	799.125	319.650
13	S	17	221	15	0.672673	0.8	1.05	0.8	1	6.7805	7.093	55.707	557.068	13.561	277.012	834.080	333.632
14	S	17	238	17	0.648204	0.8	1.05	0.8	1	7.4051	7.280	57.173	571.733	14.810	300.276	872.009	348.804
15	S	17	255	17	0.626224	0.8	1.05	0.8	1	7.1540	7.154	56.187	561.873	14.308	322.751	884.623	353.849

Daya Dukung Friksi Tiang Pancang dan Tiang Bor untuk tanah Pasir



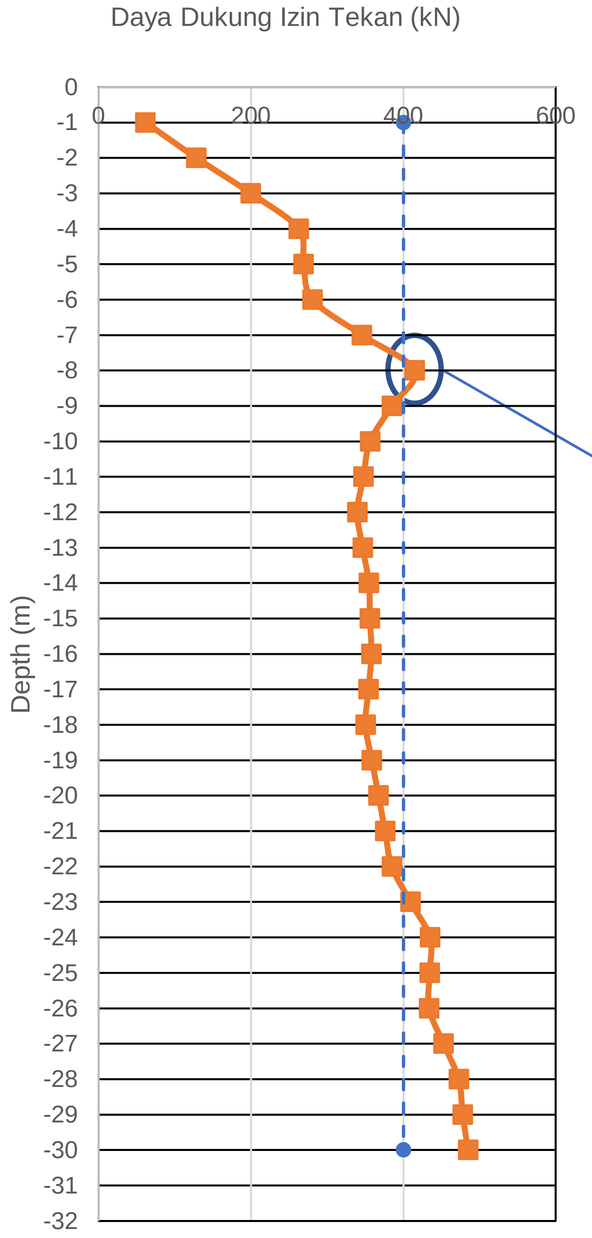
D = 0.5 m

Beban : 17 lantai (1 lantai asumsi 2 ton/m2)

Asumsi beban single p 340 kN/m^2

Untuk beban 17 lantai, dibutuhkan single pile
ukuran diameter 0.5 meter dengan Panjang tiang 8m

Solution Assignment #4



D = 0.5 m

Beban = 20 lantai (1 lantai asumsi 2 ton/m2)

Asumsi beban single pile = 400 kN/m^2

Untuk beban 20 lantai, dibutuhkan single pile ukuran diameter 0.5 meter dengan Panjang tiang 8m

Korelasi Empiris Nilai N_{-SPT} dan Berat Jenis Tanah γ untuk Tanah Pasir dan Lempung

Tabel 2.6 Korelasi empiris antara nilai N-SPT dengan *unconfined compressive strength* dan berat jenis tanah jenuh (γ_{sat}) untuk tanah kohesif.

N SPT (blows/ft)	Konsistensi	q_u (Unconfined Compressive Strength) tons / ft^2	γ_{sat} kN/ m^3
< 2	Very soft	< 0,25	16 – 19
2 – 4	Soft	0,25 – 0,50	16 – 19
4 – 8	Medium	0,50 – 1,00	17 – 20
8 – 15	Stiff	1,00 – 2,00	19 – 22
15 – 30	Very stiff	2,00 – 4,00	19 – 22
> 30	Hard	> 4,00	19 – 22

(Soil Mechanics, Lambe & Whitman, from Terzaghi and Peck 1948, Internasional Edition 1969).

Korelasi Empiris Nilai N-SPT dan Berat Jenis Tanah γ untuk Tanah Pasir dan Lempung

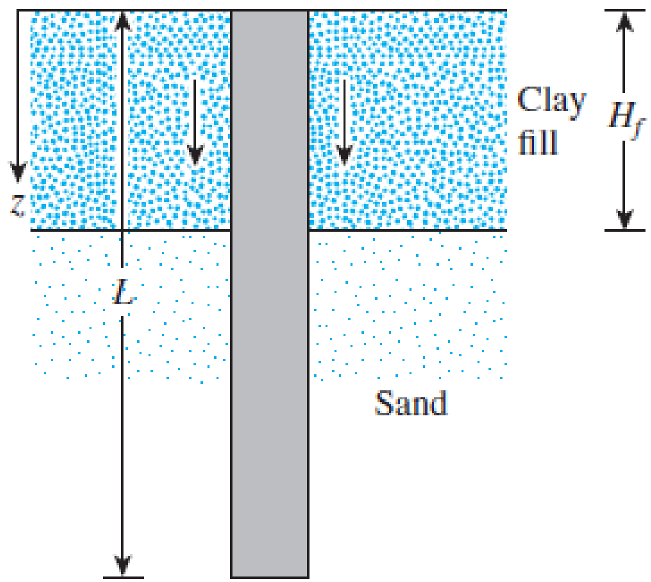
Tabel 2.7 Korelasi Berat Jenis Tanah (γ) Untuk Tanah Non Kohesif dan Kohesif.

	Cohesionless Soil				
N	0-10	11-30	31-50	>50	
Unit Weight γ, kN/m^3	12-16	14-18	16-20	18-23	
Angle of Friction φ	25-32	28-36	30-40	>35	
State	Loose	Medium	Dense	Very Dense	
	Cohesive				
N	<4	4-6	6-15	16-25	>25
Unit Weight γ, kN/m^3	14-18	16-18	16-18	16-20	>20
q_u, kPa	<25	20-50	30-60	40-200	>100
Consistency	Very Soft	Soft	Medium	Stiff	Hard

(Soil Mechanics, Whilliam T., Whitman ,Robert V., 1962)

Negative Skin Friction

Negative Skin Friction adalah gaya tarik ke bawah pada tiang akibat tanah disekitar tiang. Gaya ini terjadi akibat kondisi sebagai berikut :

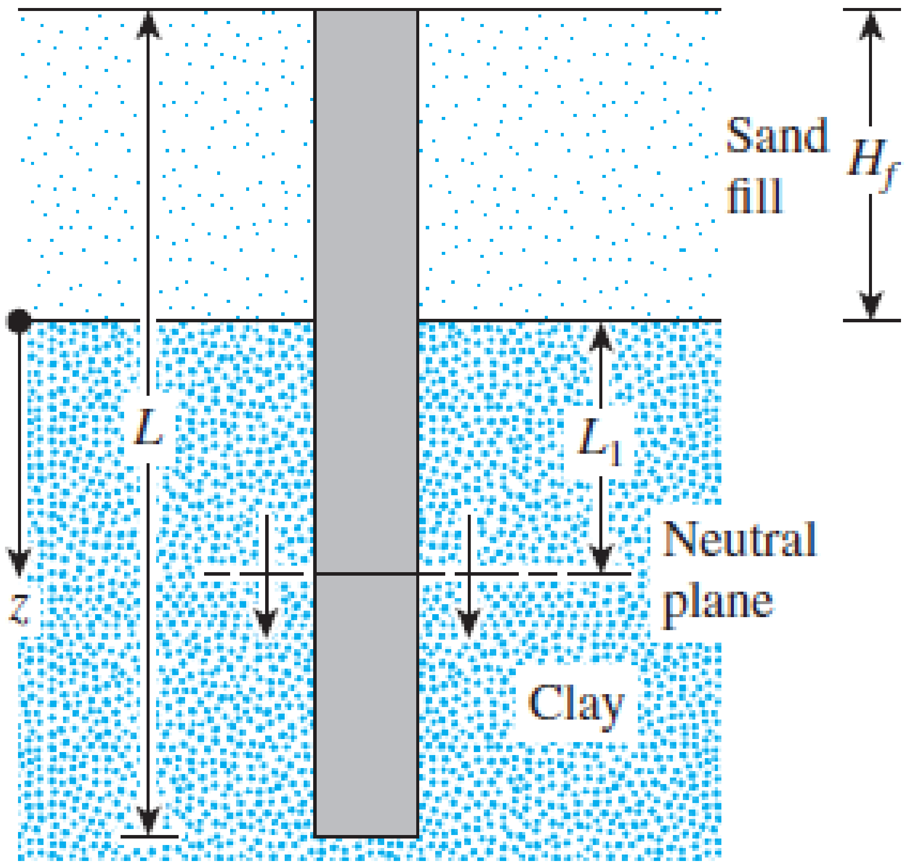


1. Jika muatan tanah lempung berada di atas lapisan tanah granular (pasir), Muatan inin secara bertahap akan mengalami konsolidasi. Proses konsolidasi ini akan menarik muatan clay ke bawah tiang selama waktu konsolidasi.

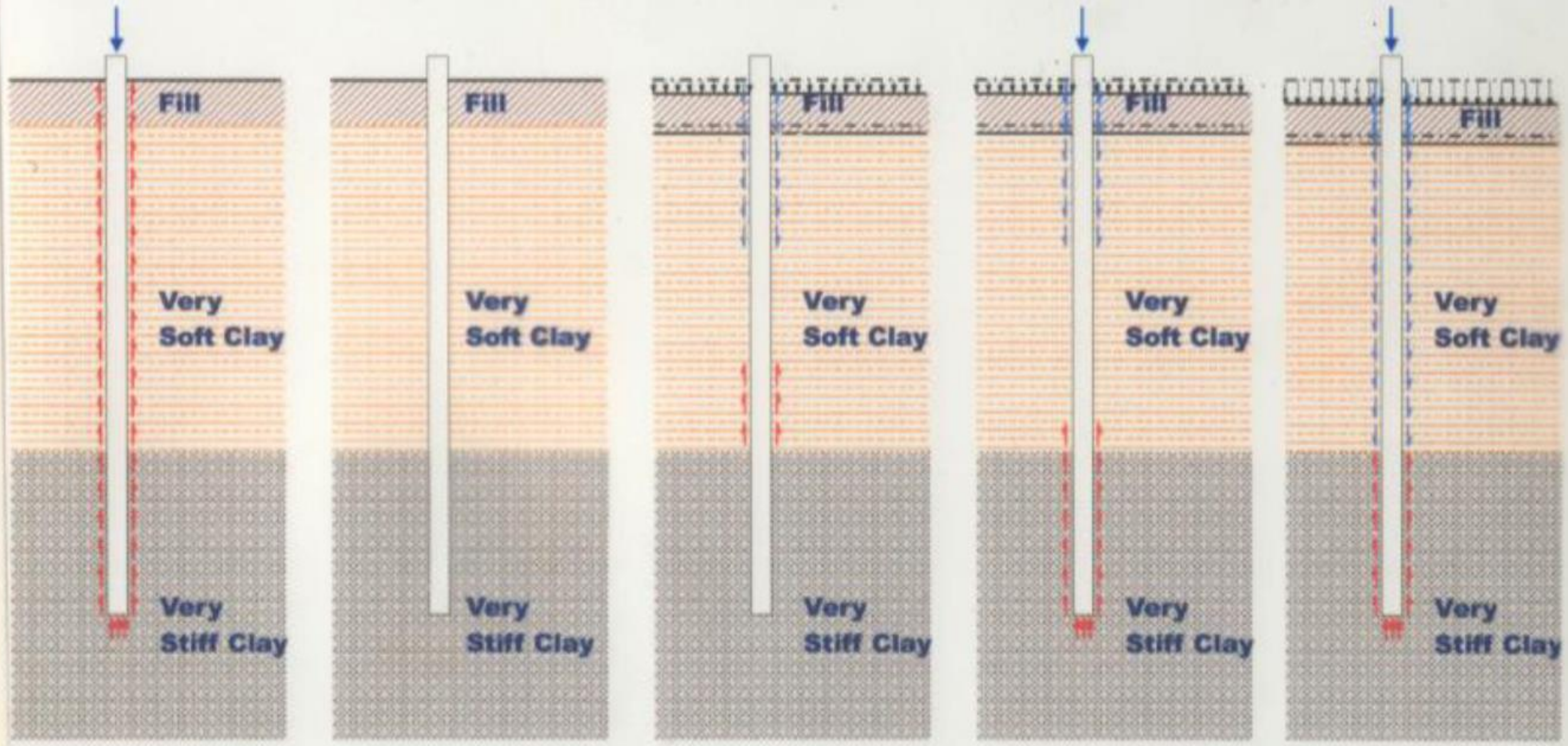
Negative Skin Friction

2. Jika muatan tanah granular berada di atas tanah lempung lunak. Akan menyebabkan proses konsolidasi pada lapisan tanah lempung sehingga lapisan tanah sand ikut bergerak ke bawah

3. Kondisi muka air tanah yang rendah dapat meningkatkan tegangan efektif vertical pada tanah di kedalaman tertentu. Yang mana akan menyebabkan penurunan konsolidasi di lempung. Jika tiang berada pada lapisan tanah lempung, akan dikenakan gaya menarik kebawah.

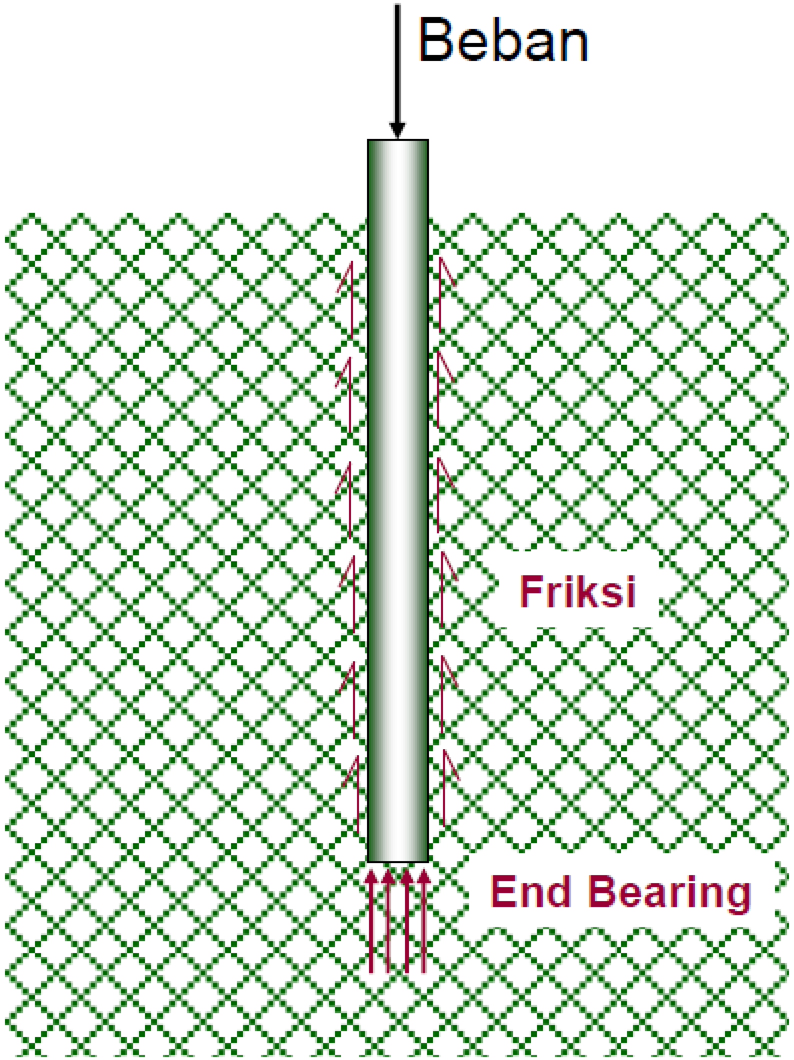


Negative Skin Friction



$$P_{all} = \frac{P_{ult}\,(\text{positif}) - P_{drain}\,(\text{negatif})}{SF}$$

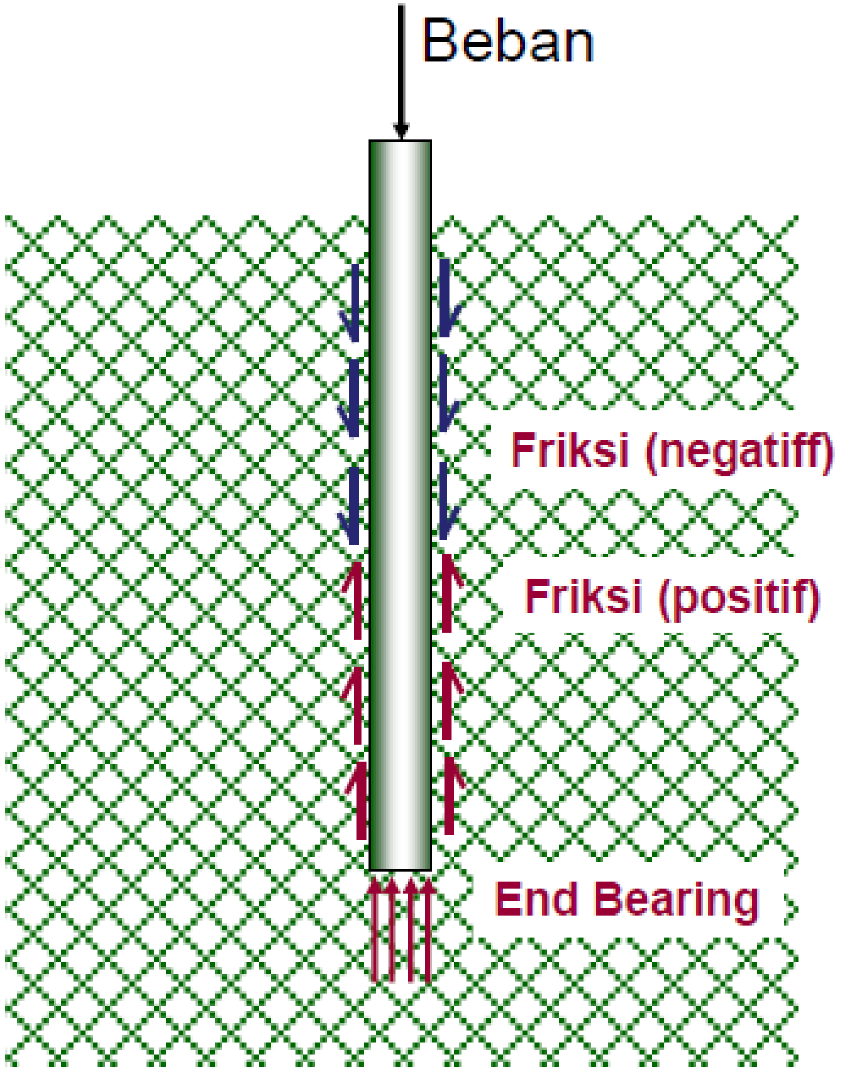
Review: Daya Dukung Aksial



$$Q_u = Q_p + Q_s$$

$$Q_{all} = \frac{Q_u}{F.S.}$$

Review: Daya Dukung Aksial



$$Q_u = Q_p + Q_s - Q_{NSF}$$

METODA PERHITUNGAN NEGATIVE SKIN FRICTION :

→ **Negative Skin Friction pada Kondisi Un-Drained :**

$$P_{friction} = \sum_{x=Lf+Lc}^{x=Lf+Lc+Lb} \tau \Psi_{area\ selimut}$$

$$= \pi\phi_{pile\ diameter} \left(\Sigma \alpha\, Cu + \Sigma\, 0{,}5 Ks\, \sigma_v \tan\delta\ \Delta z\right)$$

tanah lempung　　　　tanah pasir

$$f = K\sigma'_o \tan\delta'$$

dimana,

- α = faktor adhesi
- Cu = undrained shear strength dari nilai N - SPT
- Ks = Koeffisien lateral earth pressure. $K_o \approx 1 - \sin\phi'$
- δ = interface sudut geser dalam antar tiang dan tanah.
- σ_v = effective overbourden pressure.

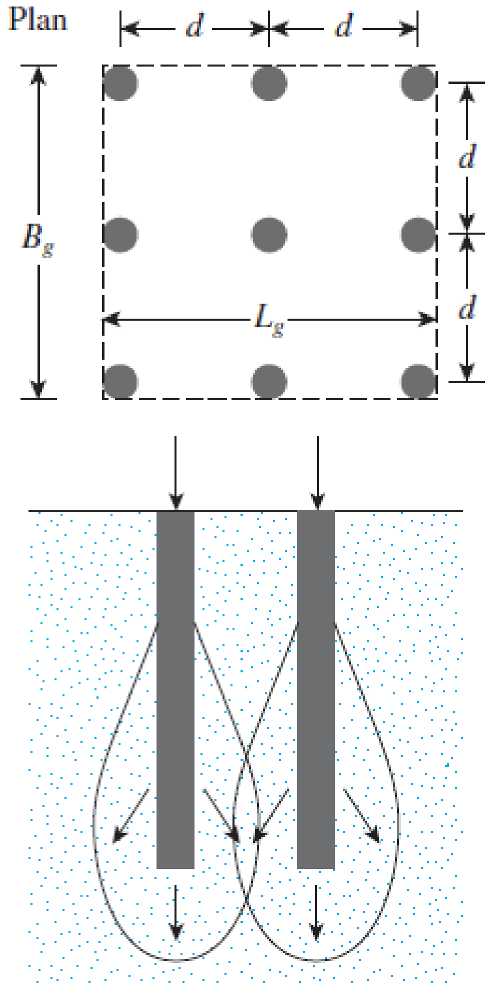
Soil	δ, degrees	Limiting f, kips/ft² (kPa)
very loose to medium, sand to silt	15	1.0 (47.8)
loose to dense, sand to silt	20	1.4 (67.0)
medium to dense, sand to sand-silt	25	1.7 (83.1)
dense to very dense, sand to sand-silt	30	2.0 (95.5)
dense to very dense, gravel to sand	35	2.4 (114.8)

Prakash dan Sharma, 1990

Group Pile Efficiency

In most cases, piles are used in groups to transmit the structural load to the soil. *A pile cap* is constructed over *group piles*. The cap can be in contact with the ground, as in most cases, or well above the ground, as in the case of offshore platforms.

When the piles are placed close each other, a reasonable assumption is that the stresses transmitted by the piles to soil will overlap and reducing the load-bearing capacity. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles. In practice, the minimum center-to-center pile spacing, *d*, is *2.5D* and, in ordinary situations, is actually about 3 to 3.5*D*.



Group Pile Efficiency

The efficiency of the load-bearing capacity of a group pile may be defined as :

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u}$$

where

η = group efficiency
$Q_{g(u)}$ = ultimate load-bearing capacity of the group pile
Q_u = ultimate load-bearing capacity of each pile without the group effect

Group Pile Efficiency

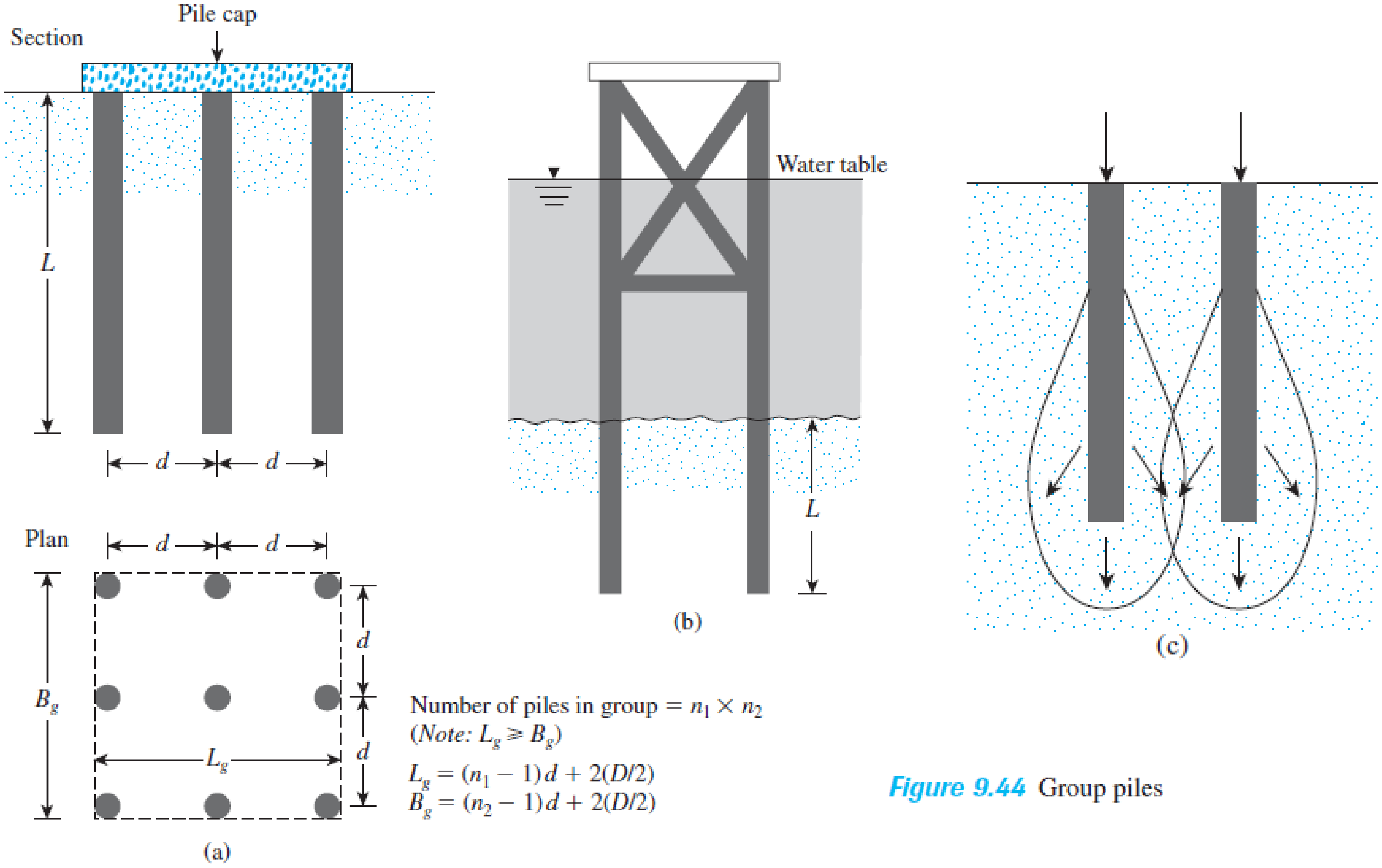
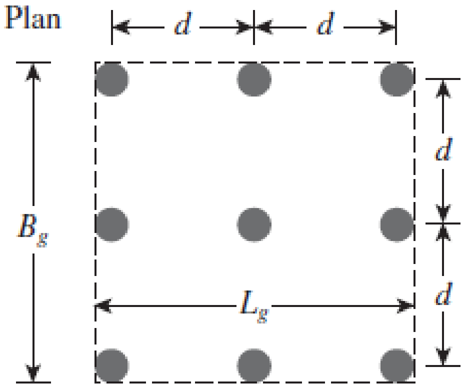
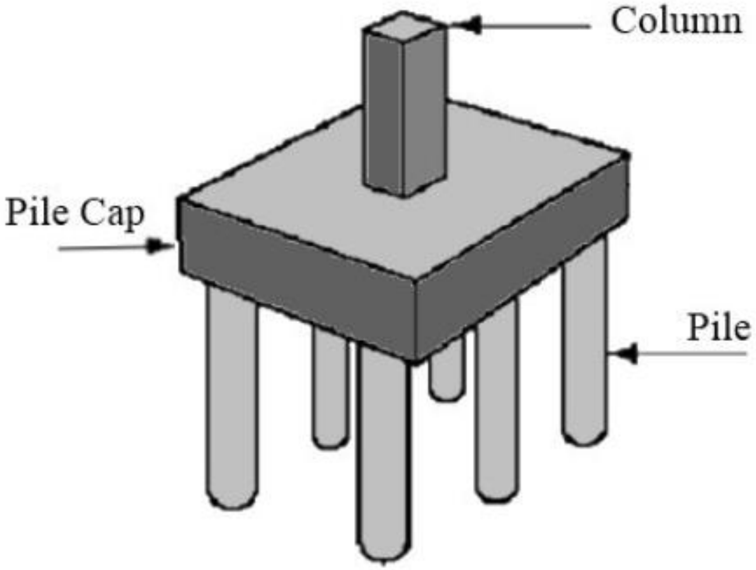


Figure 9.44 Group piles

Group Pile Efficiency





Perimeter blok Pile group
Jumlah tiang arah L = n1
Jumlah tiang arah B = n2

2 sisi Lg = 2 {(n1-1)d + D}
2 sisi Bg = 2 {(n2-1)d + D}

2 Sisi Lg+ 2 sisi Bg =
2 {(n1-1)d + D} + 2 {(n2-1)d + D}

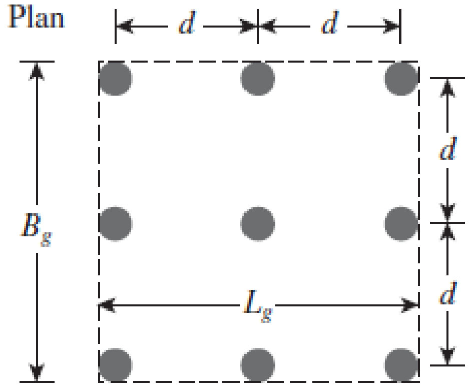
2 {(n1+n2-2d) + 2D}

$$2(n_1 + n_2 - 2)d + 4D,$$

Group Pile Efficiency

Many structural engineers use a simplified analysis to obtain the group efficiency for *friction piles,* particularly in sand. This type of analysis can be explained with the aid of Figure 9.44a. Depending on their spacing within the group, the piles may act in one of two ways: (1) as a *block,* with dimensions $L_g \times B_g \times L$, or (2) as *individual piles*. If the piles act as a block, the frictional capacity is $f_{av} p_g L \approx Q_{g(u)}$. [*Note:* p_g = perimeter of the cross section of block $= 2(n_1 + n_2 - 2)d + 4D$, and f_{av} = average unit frictional resistance.]

Group Pile Efficiency



Similarly, for each pile acting individually, $Q_u \approx pLf_{av}$. (*Note:* p = perimeter of the cross section of each pile.) Thus,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{f_{av}[2(n_1 + n_2 - 2)d + 4D]L}{n_1 n_2 pLf_{av}} = \frac{2(n_1 + n_2 - 2)d + 4D}{pn_1 n_2} \tag{9.128}$$

Hence,

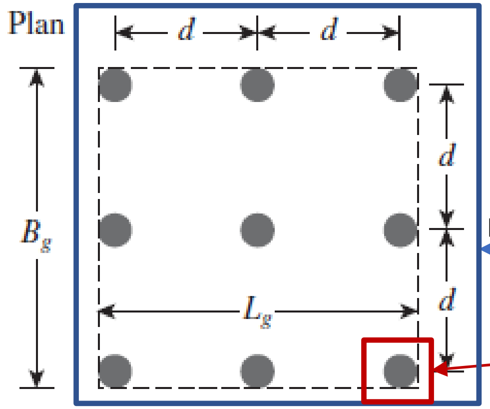
$$Q_{g(u)} = \left[\frac{2(n_1 + n_2 - 2)d + 4D}{pn_1 n_2}\right]\Sigma Q_u \tag{9.129}$$

From Eq. (9.129), if the center-to-center spacing d is large enough, $\eta > 1$. In that case, the piles will behave as individual piles. Thus, in practice, if $\eta < 1$, then

$$Q_{g(u)} = \eta \Sigma Q_u$$

and if $\eta \geq 1$, then

$$Q_{g(u)} = \Sigma Q_u$$



Similarly, for each pile acting individually, $Q_u \approx pLf_{av}$. (*Note:* p = perimeter of the cross section of each pile.) Thus,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{f_{av}[2(n_1 + n_2 - 2)d + 4D]L}{n_1 n_2 p L f_{av}} = \frac{\boxed{2(n_1 + n_2 - 2)d + 4D}}{\boxed{pn_1 n_2}} \qquad (9.128)$$

Keliling blok tiang (blok group pile)

Jumlah seluruh Keliling satu tiang

Hence,

$$Q_{g(u)} = \left[\frac{2(n_1 + n_2 - 2)d + 4D}{pn_1 n_2}\right]\Sigma Q_u \qquad (9.129)$$

From Eq. (9.129), if the center-to-center spacing d is large enough, $\eta > 1$. In that case, the piles will behave as individual piles. Thus, in practice, if $\eta < 1$, then

$$Q_{g(u)} = \eta \Sigma Q_u$$

and if $\eta \geq 1$, then

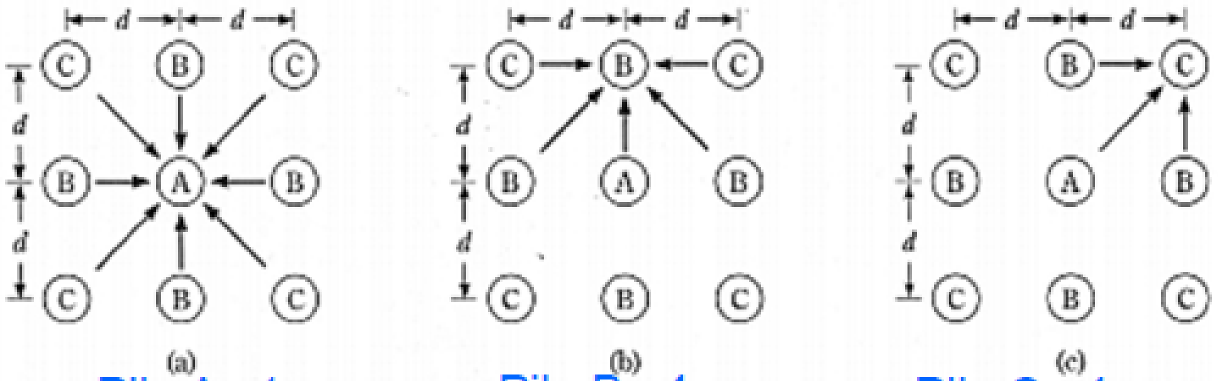
$$Q_{g(u)} = \Sigma Q_u$$

Group Pile Efficiency

Table 9.18 Equations for Group Efficiency of Friction Piles

Name	Equation
Converse–Labarre equation	$\eta = 1 - \left[\frac{(n_1 - 1)n_2 + (n_2 - 1)n_1}{90 n_1 n_2}\right]\theta$ where $\theta(\text{deg}) = \tan^{-1}(D/d)$
Los Angeles Group Action equation	$\eta = 1 - \frac{D}{\pi d n_1 n_2}[n_1(n_2 - 1) + n_2(n_1 - 1) + \sqrt{2}(n_1 - 1)(n_2 - 1)]$
Seiler–Keeney equation (Seiler and Keeney, 1944)	$\eta = \left\{1 - \left[\frac{11d}{7(d^2 - 1)}\right]\left[\frac{n_1 + n_2 - 2}{n_1 + n_2 - 1}\right]\right\} + \frac{0.3}{n_1 + n_2}$ where d is in ft

Feld (1943) suggested a method by which the load capacity of individual piles (when only frictional resistance is considered) in a group embedded in sand could be assigned. According to this method, the ultimate capacity of a pile is reduced by one-sixteenth by each adjacent diagonal or row pile.



Pile A : 1
reduction factor:

$1 - \frac{8}{16} \Rightarrow 0.5Q_u$

Pile B : 4
reduction factor:

$1 - \frac{5}{16} \Rightarrow 2.75Q_u$

Pile C : 4
reduction factor:

$1 - \frac{3}{16} \Rightarrow 3.25Q_u$

Hence,

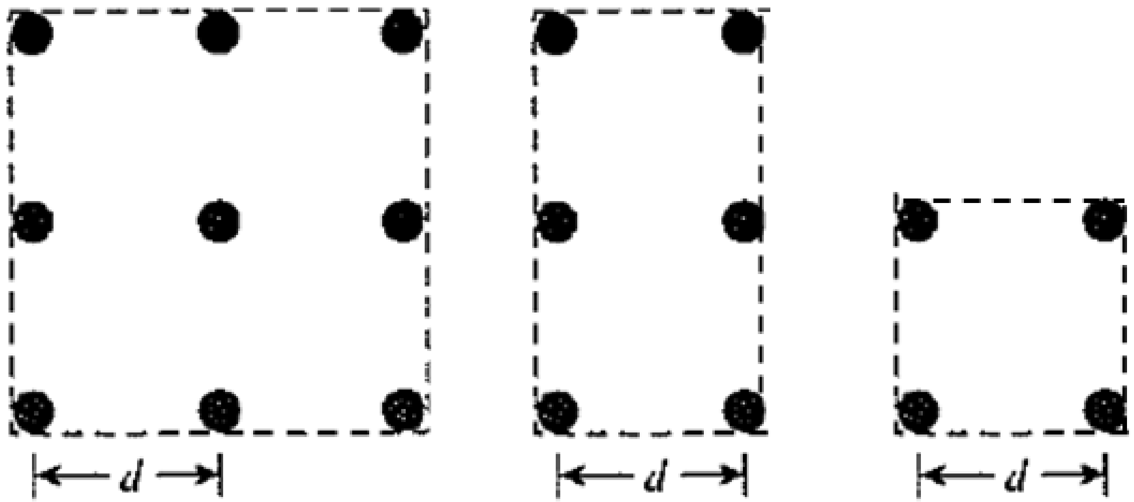
$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u} = \frac{6.5Q_u}{9Q_u} = 72\%$$

Group Pile Efficiency

Problem No 4

The plan of a group pile (*friction pile*) in sand is shown in figure. The piles are circular in cross section and have an outside diameter of 450 mm. The center-to-center spacing pf the piles (d) are 900 mm. Find the efficiency of the pile group using:

a) Simplified Equation.
b) Converse-Labarre Equation.
c) Fled Equation.



(1) (2) (3)

Problem No 4

The plan of a group pile (*friction pile*) in sand is shown in figure. The piles are circular in cross section and have an outside diameter of 450 mm. The center-to-center spacing pf the piles (*d*) are 900 mm. Find the efficiency of the pile group using:

a) Simplified Equation.

$$\left[\frac{2(n_1 + n_2 - 2)d + 4D}{p n_1 n_2}\right]$$

$n_1 = 2$
$n_2 = 3$
$d = 0.9$ m
$D = 0.45$ m

Perimeter single pile = $\pi \cdot D$

$$\mu = \frac{2(2+3-2)0.9+(4 * 0.45)}{\underbrace{3.14 * 0.45}_{\pi \cdot D} * 2 * 3} = 84{,}9\ \%$$

Misal masing-masing tiang memiliki daya dukung 300kN.
Jumlah daya dukung seluruh tiang = 2 x 3 x 300 kN= 1800kN
Daya dukung pile group setelah dikali dengan efisiensi group tiang
1800kN x μ = 1800 x 84.9 % = 1528 kN

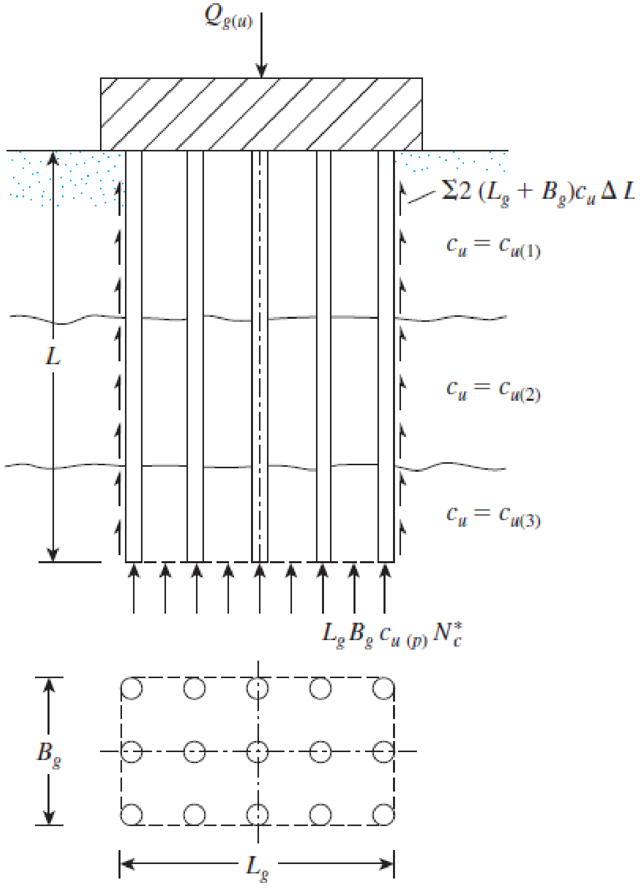
Ultimate Capacity in Saturated Clays

Step 1: Determine $\Sigma Q_u = n_1 n_2 (Q_p + Q_s)$. $Q_p = A_p[9c_{u(p)}]$

where $c_{u(p)}$ = undrained cohesion of the clay at the pile tip

$$Q_s = \Sigma \alpha p c_u \Delta L$$

So, $\Sigma Q_u = n_1 n_2 [9 A_p c_{u(p)} + \Sigma \alpha p c_u \Delta L]$



Step 2: Determine the ultimate capacity by assuming that the piles in the group act as a block with dimensions L_g x B_g x L. The skin resistance of the block is

$$\Sigma p_g c_u \Delta L = \Sigma\, 2(L_g + B_g) c_u \Delta L$$

$$A_p q_p = A_p c_{u(p)} N_c^* = (L_g B_g) c_{u(p)} N_c^*$$

$$\Sigma Q_u = L_g B_g c_{u(p)} N_c^* + \Sigma\, 2(L_g + B_g) c_u \Delta L$$

Ultimate Capacity in Saturated Clays

Step 3: Compare the values of ΣQ_u from step 1 and step 2. The lower of these is $Q_g(u)$

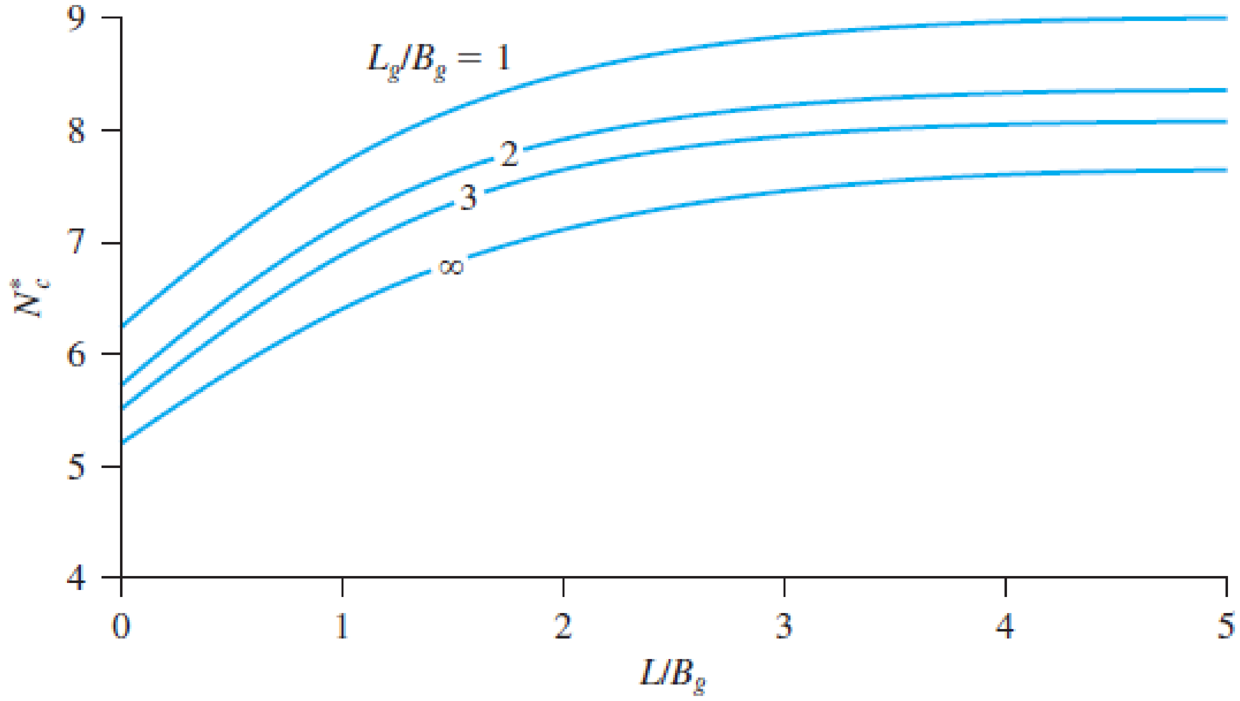
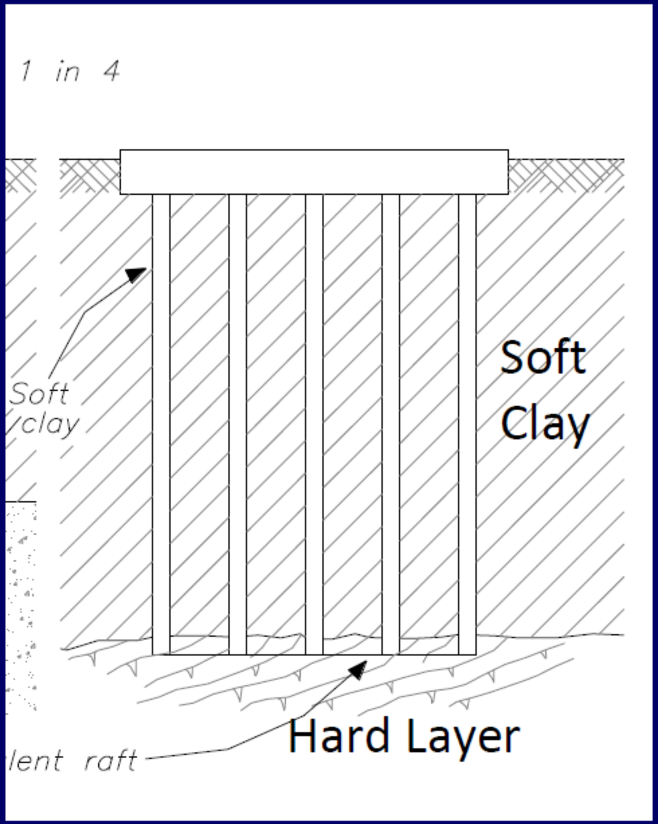
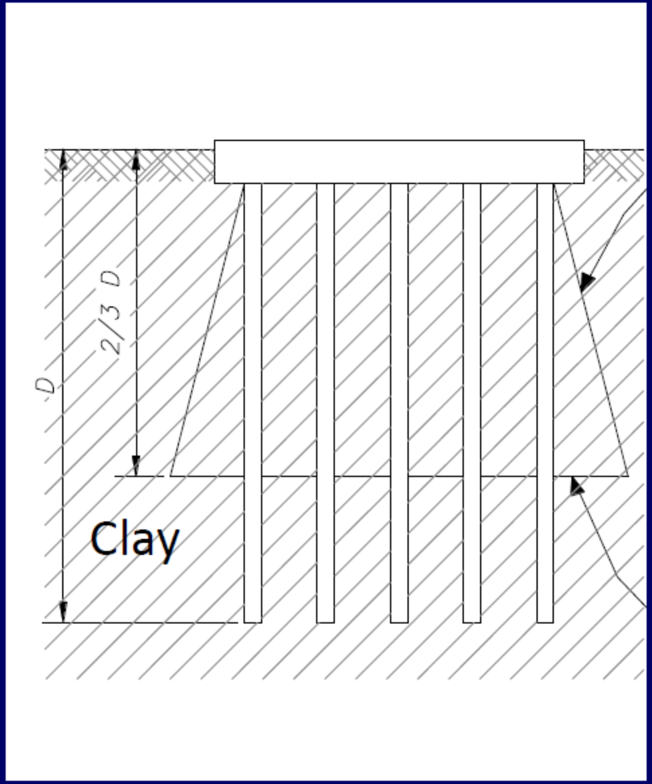


Figure 9.48 Variation of N_c^* with L_g/B_g and L/B_g

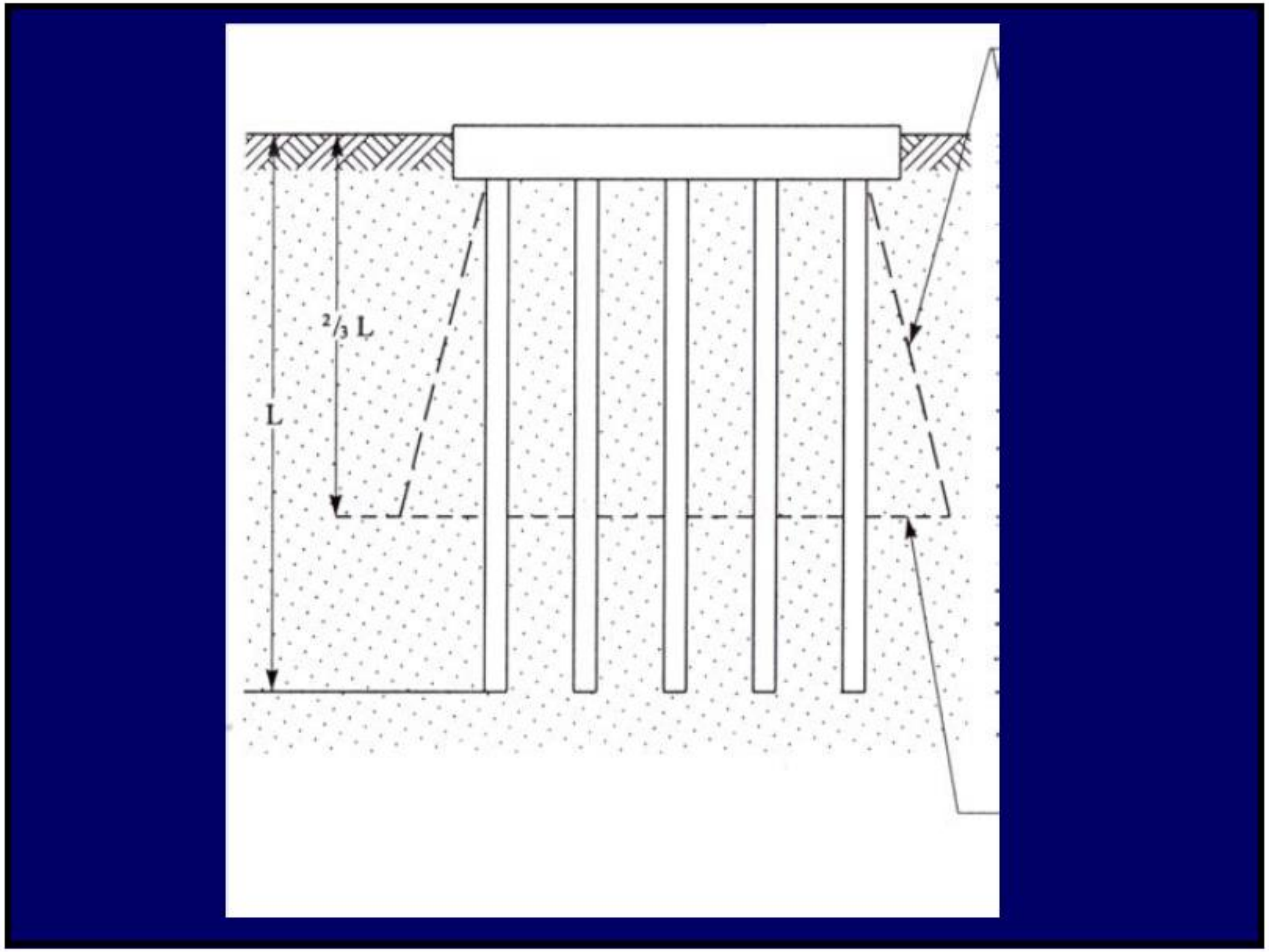
Transfer beban pada Group Tiang:





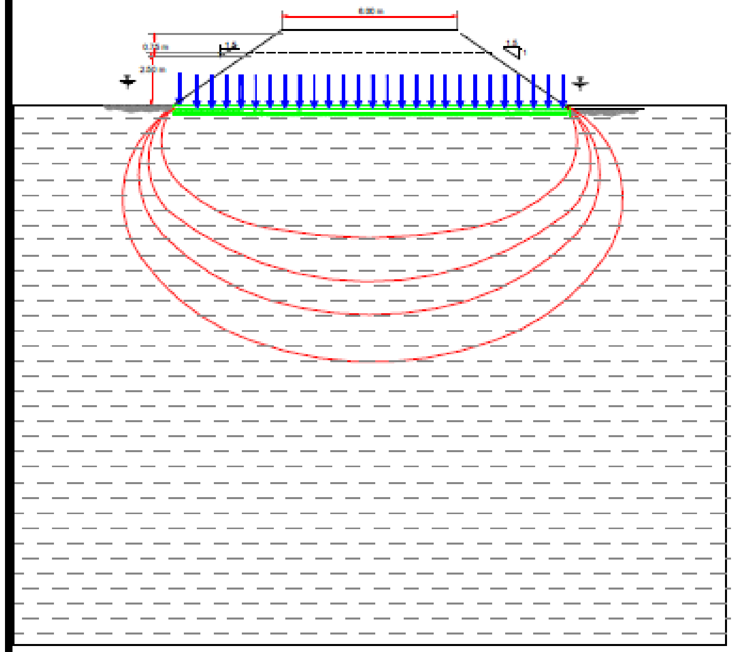
(Tomlinso, 1977)

Settlement on Group Pile

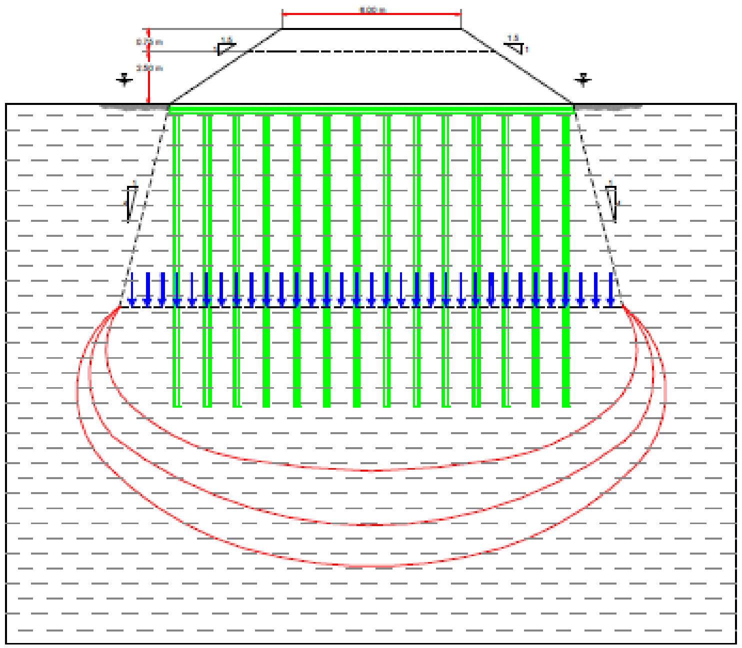


Settlement on Group Pile

Settlemet Pondasi:

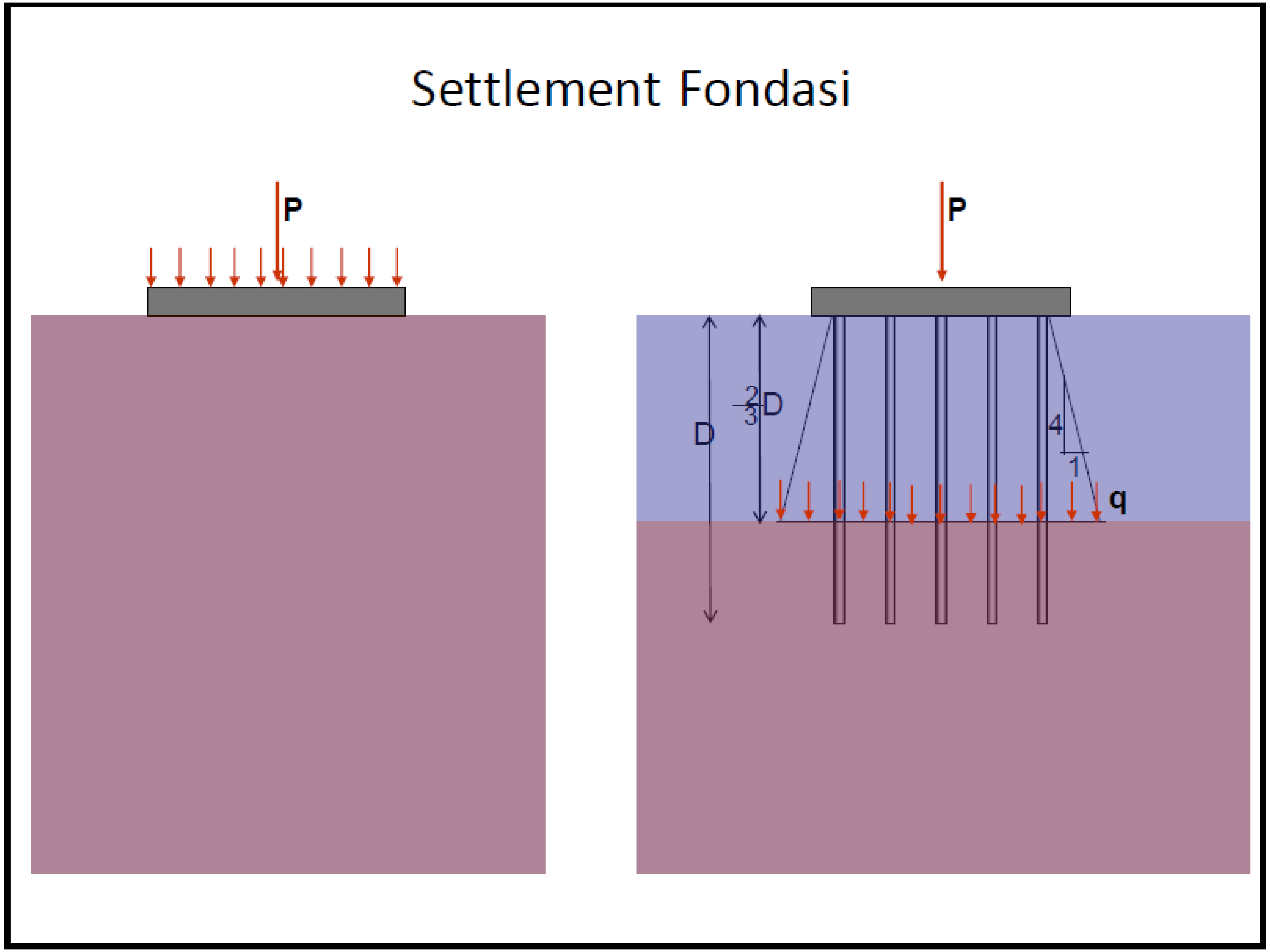


Pondasi Dangkal



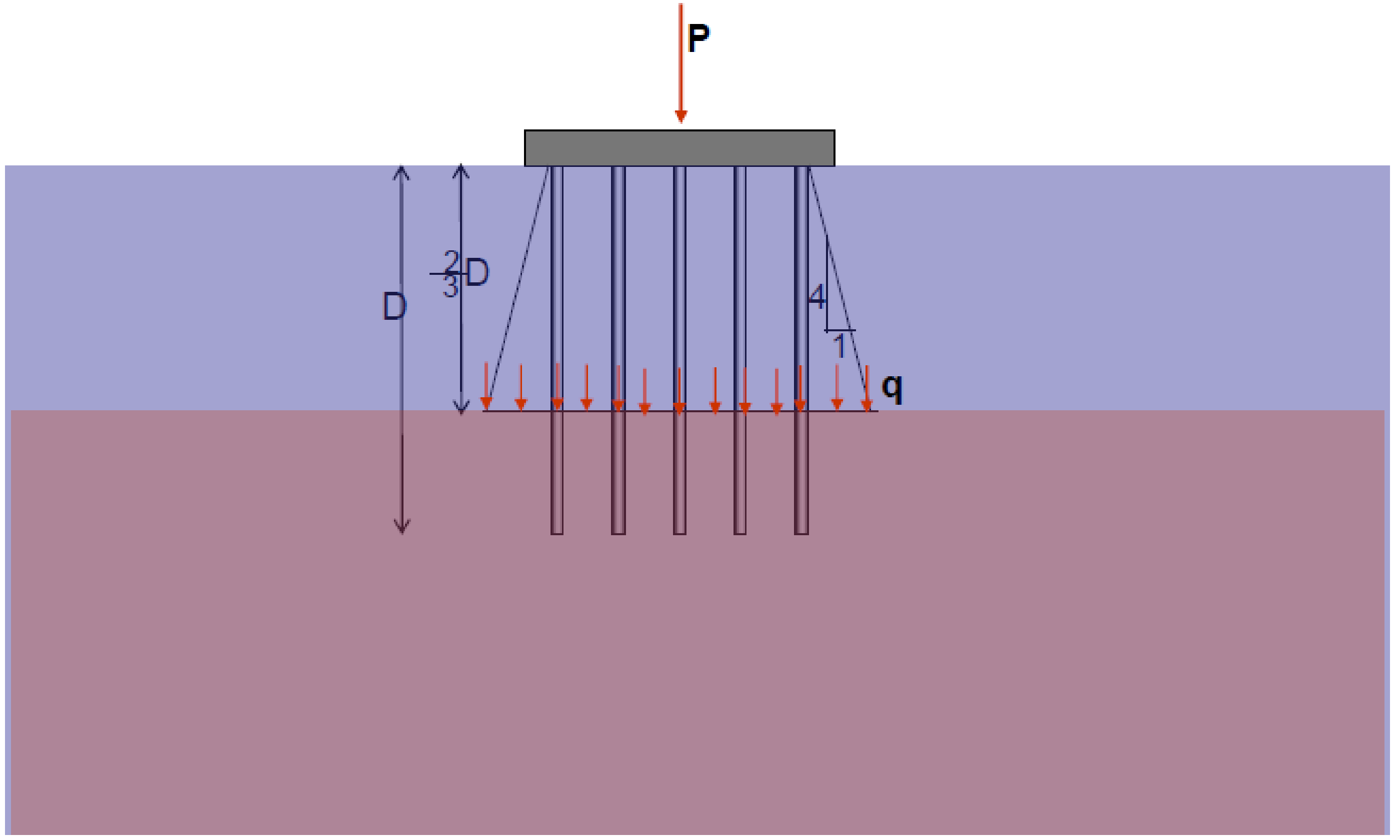
Pondasi Tiang

Settlement on Group Pile



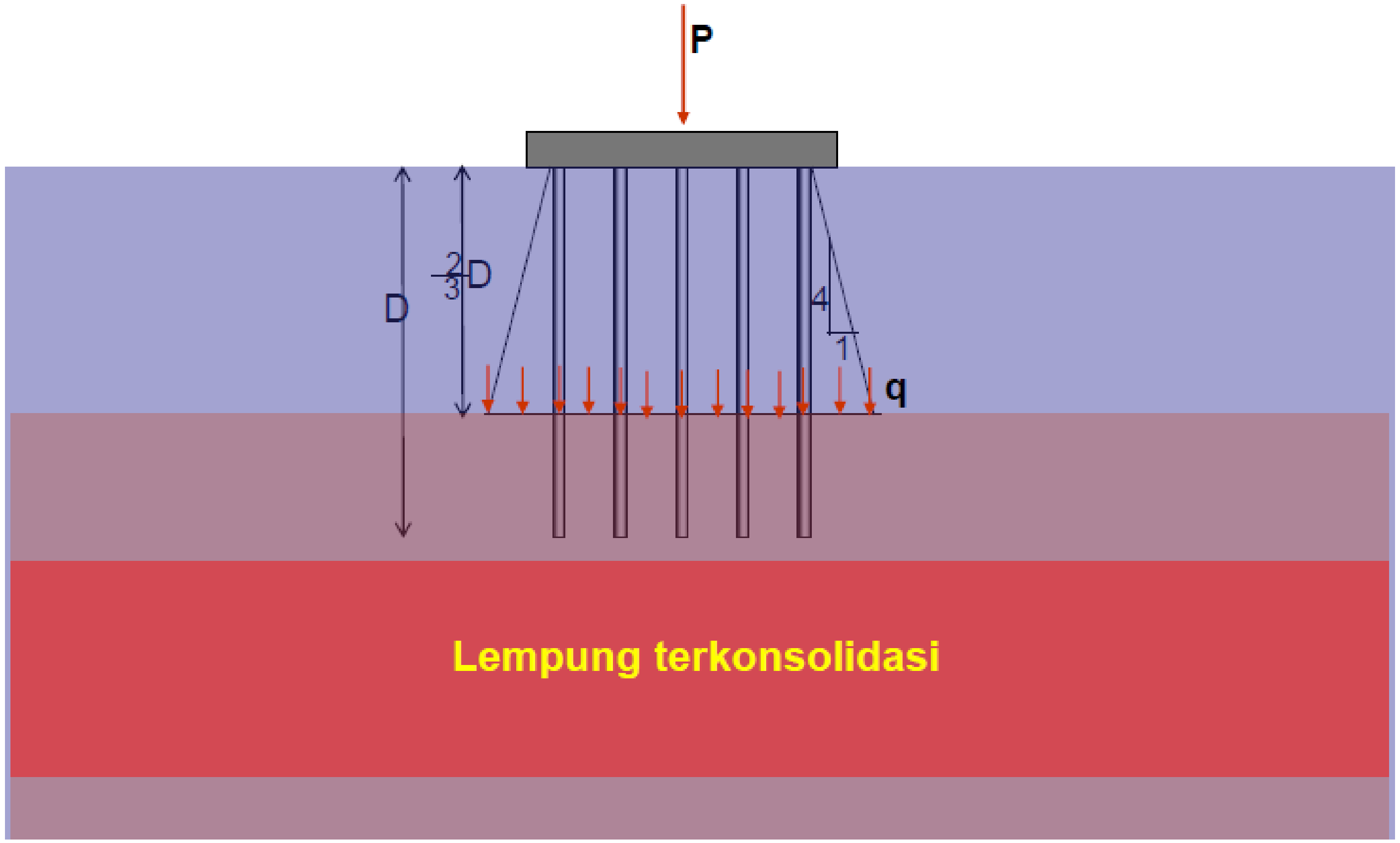
Settlement on Group Pile

Settlement Grup Tiang



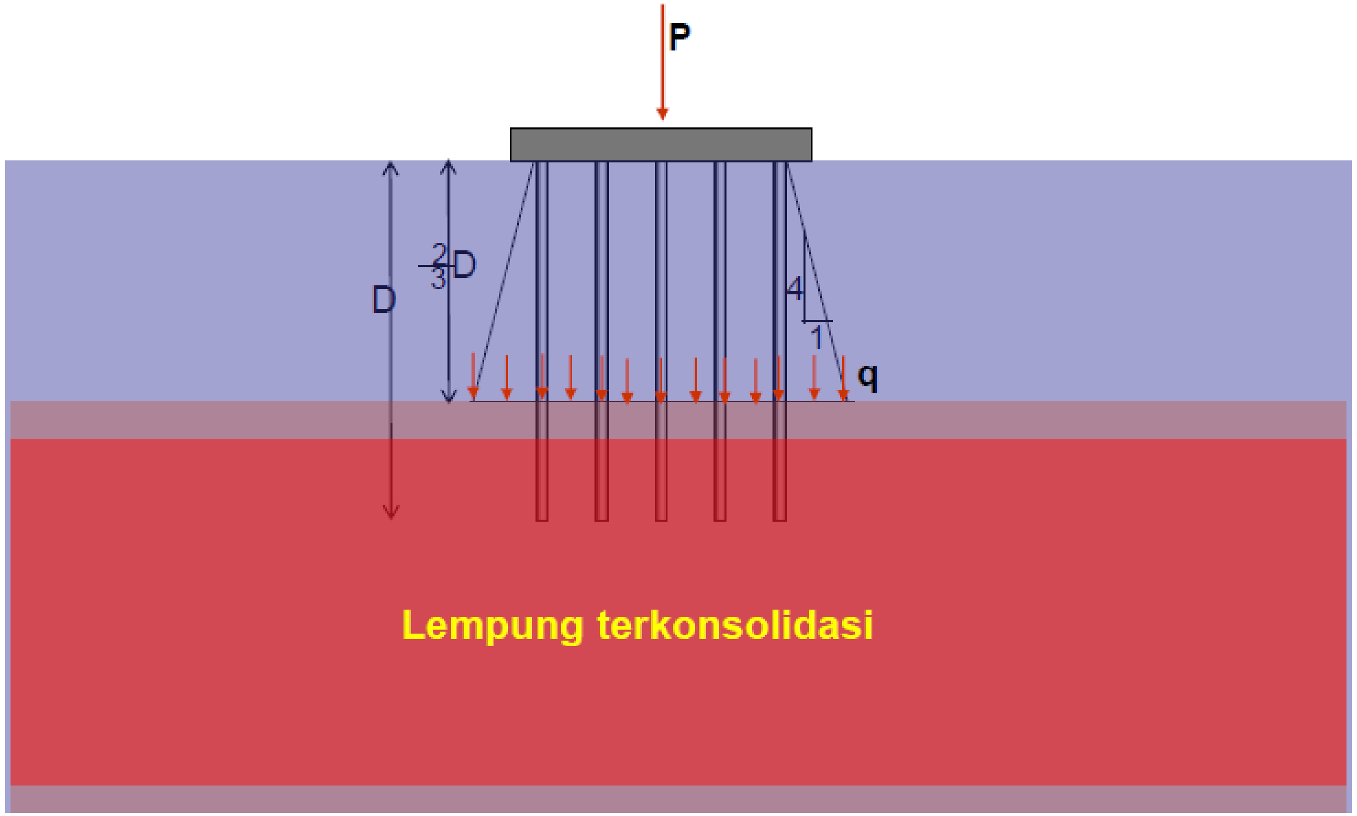
Settlement on Group Pile

Settlement Grup Tiang Di Tanah Lempung



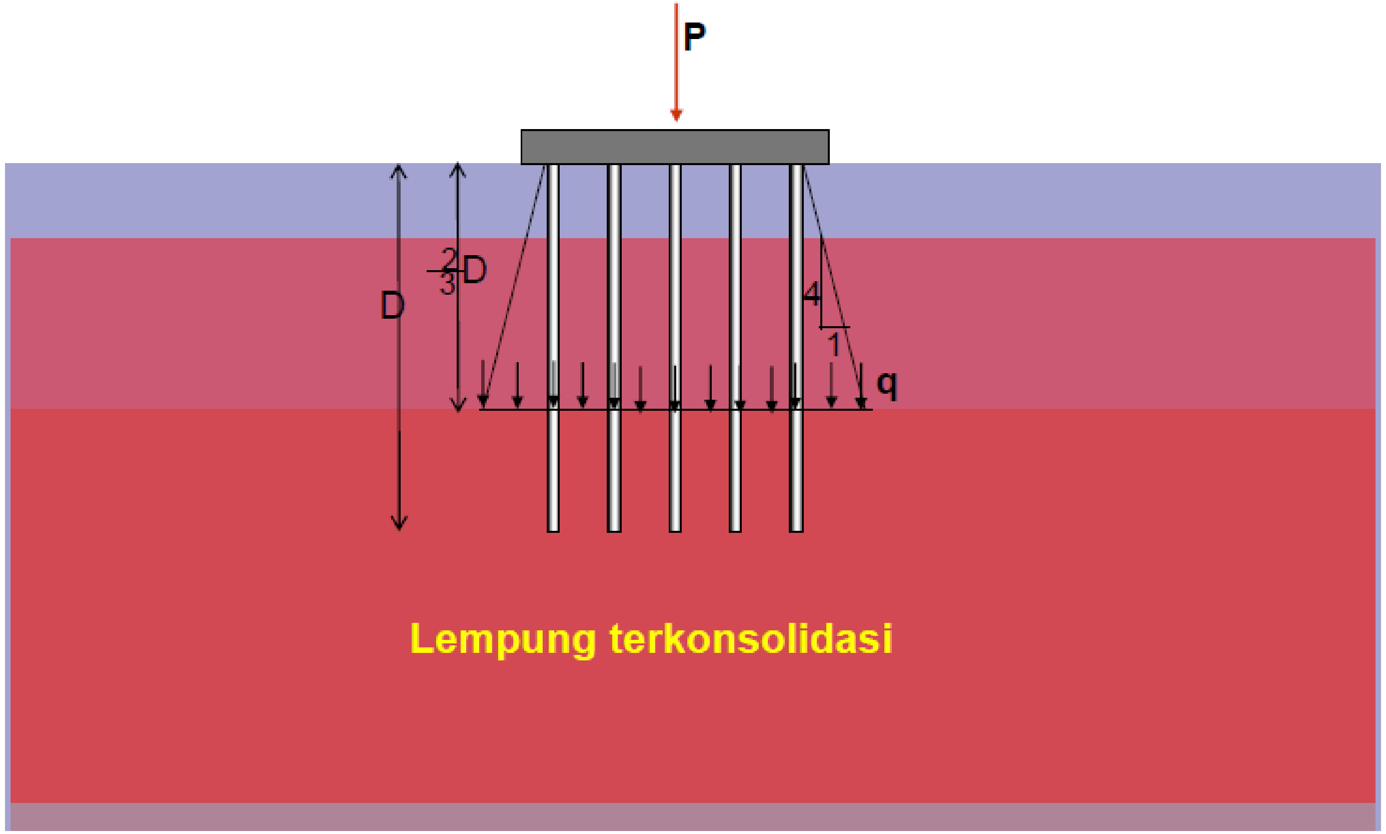
Settlement on Group Pile

Settlement Grup Tiang Di Tanah Lempung

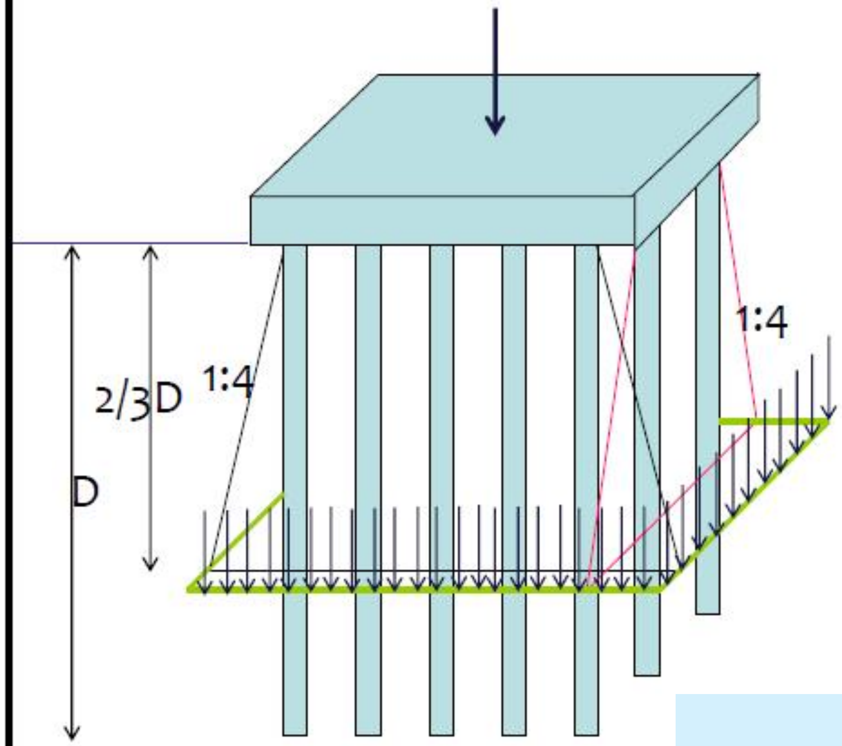


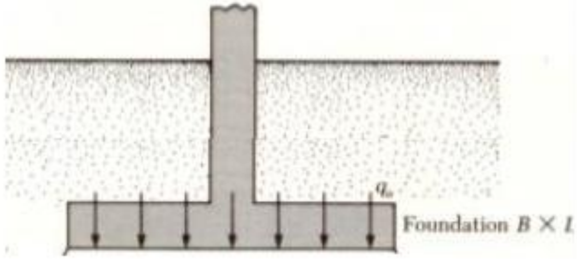
Settlement on Group Pile

Settlement Grup Tiang Di Tanah Lempung



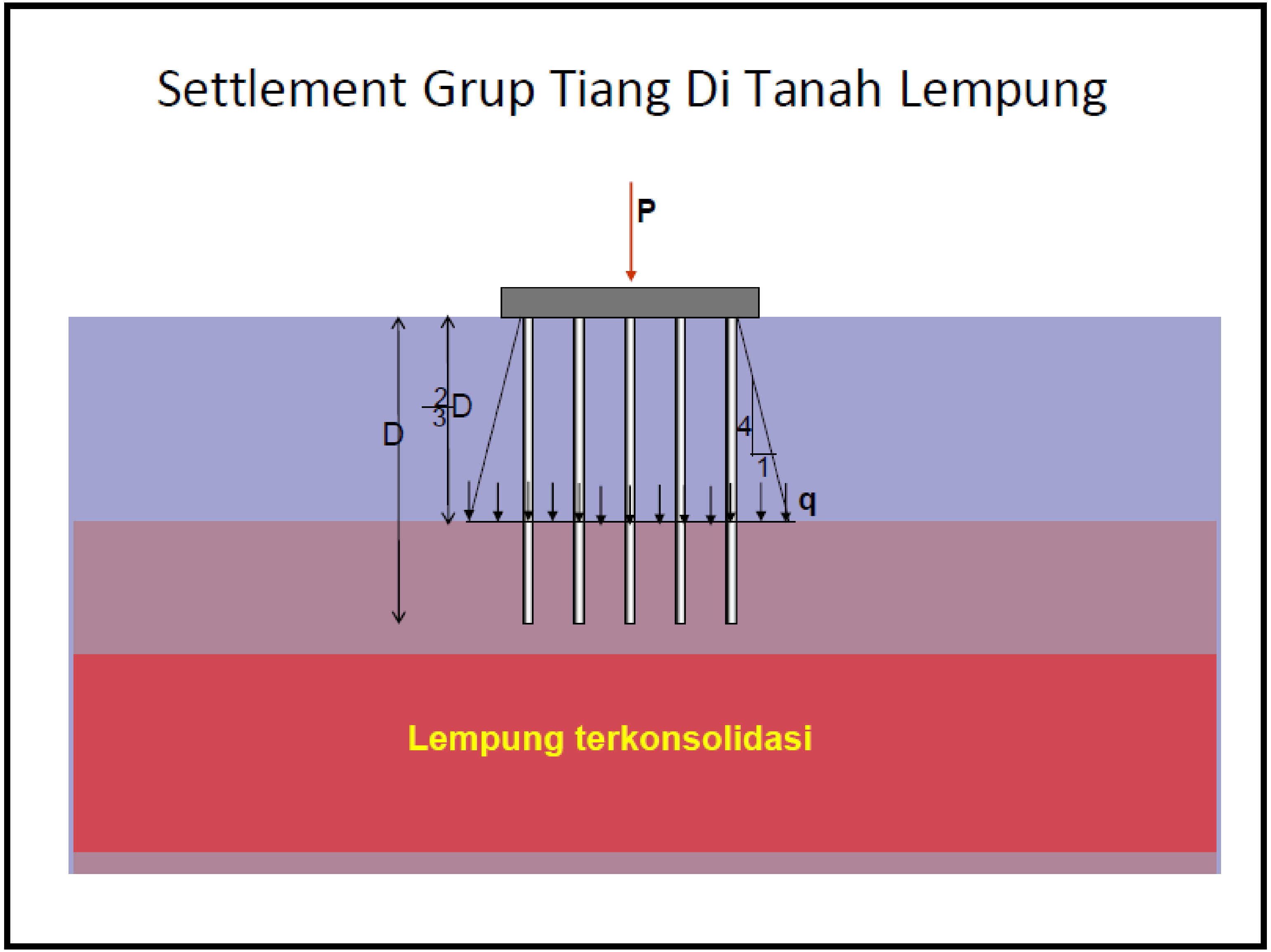
Settlement on Group Pile



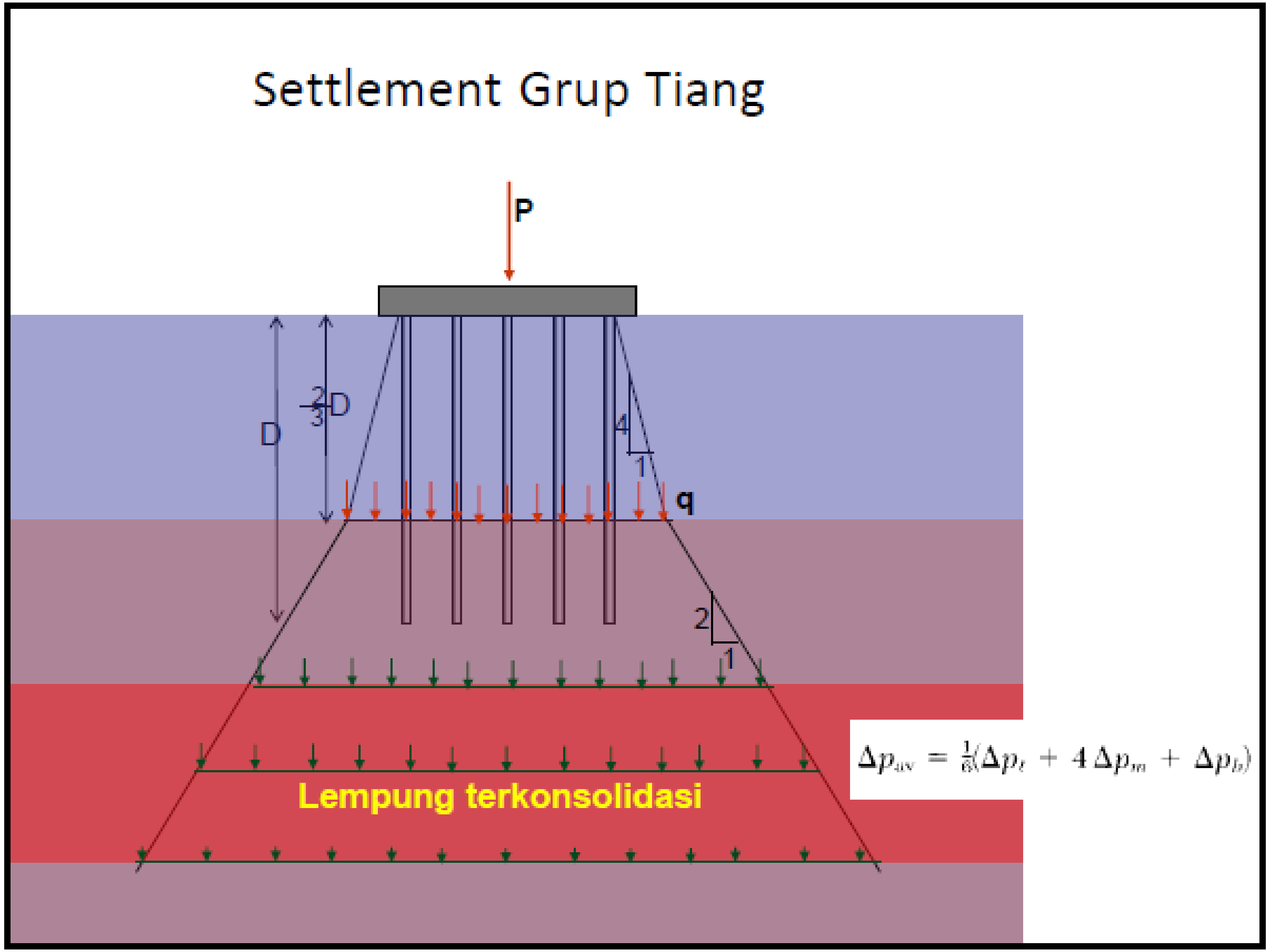


$$\Delta s_{c(1)} = \left[\frac{(C_{c(1)} H_1)}{1 + e_{o(1)}}\right] \log\left[\frac{\sigma'_{o(1)} + \Delta\sigma'_{(1)}}{\sigma'_{o(1)}}\right]$$

Settlement on Group Pile



Settlement on Group Pile



Settlement on Group Pile

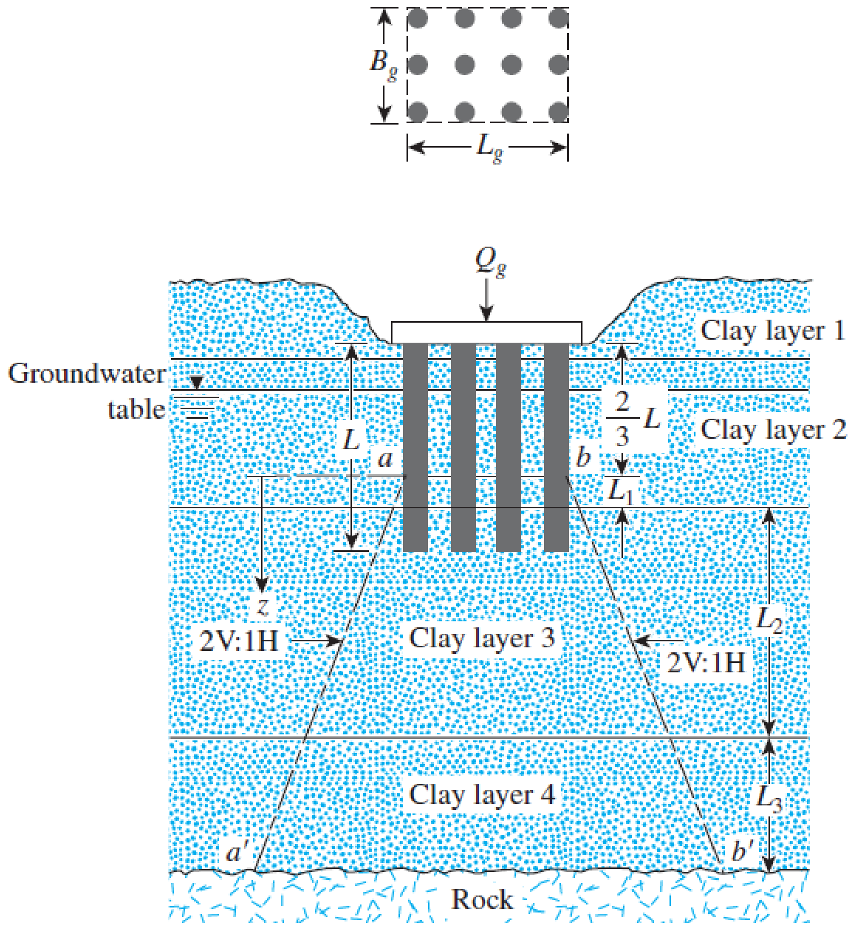


Figure 9.50 Consolidation settlement of group piles

$$\Delta s_{c(1)} = \left[\frac{(C_{c(1)}H_1)}{1 + e_{o(1)}}\right]\log\left[\frac{\sigma'_{o(1)} + \Delta\sigma'_{(1)}}{\sigma'_{o(1)}}\right]$$

where

$\Delta\sigma'_i$ = increase in effective stress at the middle of layer i
L_g, B_g = length and width, respectively of the planned group piles
z_i = distance from $z = 0$ to the middle of the clay layer i

Example 9.23

A group pile in clay is shown in Figure 9.51. Determine the consolidation settlement of the piles. All clays are normally consolidated.

Solution

Because the lengths of the piles are 15 m each, the stress distribution starts at a depth of 10 m below the top of the pile. We are given that $Q_g = 2000$ kN.

Calculation of Settlement of Clay Layer 1

For normally consolidated clays,

$$\Delta s_{c(1)} = \left[\frac{(C_{c(1)}H_1)}{1 + e_{o(1)}}\right]\log\left[\frac{\sigma'_{o(1)} + \Delta\sigma'_{(1)}}{\sigma'_{o(1)}}\right]$$

$$\Delta\sigma'_{(1)} = \frac{Q_g}{(L_g + z_1)(B_g + z_1)} = \frac{2000}{(3.3 + 3.5)(2.2 + 3.5)} = 51.6 \text{ kN/m}^2$$

and

$$\sigma'_{o(1)} = 2(16.2) + 12.5(18.0 - 9.81) = 134.8 \text{ kN/m}^2$$

So

$$\Delta s_{c(1)} = \frac{(0.3)(7)}{1 + 0.82}\log\left[\frac{134.8 + 51.6}{134.8}\right] = 0.1624 \text{ m} = \mathbf{162.4\ mm}$$

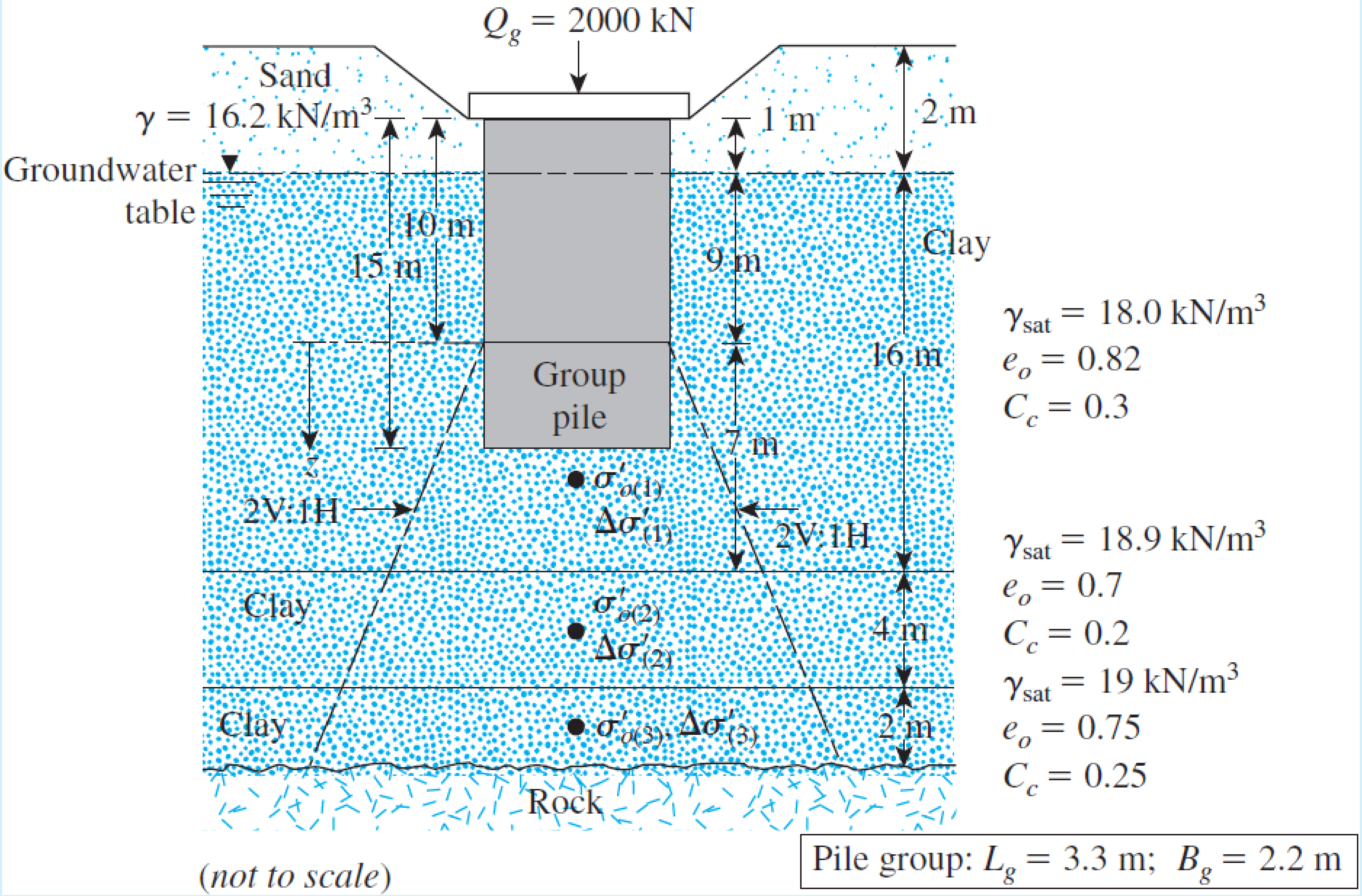


Figure 9.51 Consolidation settlement of a pile group

Settlement of Layer 2

As with layer 1,

$$\Delta s_{c(2)} = \frac{C_{c(2)}H_2}{1 + e_{o(2)}} \log\left[\frac{\sigma'_{o(2)} + \Delta\sigma_{(2)}}{\sigma'_{o(2)}}\right]$$

$$\sigma'_{\sigma(2)} = 2(16.2) + 16(18.0 - 9.81) + 2(18.9 - 9.81) = 181.62 \text{ kN/m}^2$$

and

$$\Delta\sigma'_{(2)} = \frac{2000}{(3.3 + 9)(2.2 + 9)} = 14.52 \text{ kN/m}^2$$

Hence,

$$\Delta s_{c(2)} = \frac{(0.2)(4)}{1 + 0.7} \log\left[\frac{181.62 + 14.52}{181.62}\right] = 0.0157 \text{ m} = \mathbf{15.7\ mm}$$

Settlement of Layer 3

Continuing analogously, we have

$$\sigma'_{o(3)} = 181.62 + 2(18.9 - 9.81) + 1(19 - 9.81) = 208.99 \text{ kN/m}^2$$

$$\Delta\sigma'_{(3)} = \frac{2000}{(3.3 + 12)(2.2 + 12)} = 9.2 \text{ kN/m}^2$$

$$\Delta s_{c(3)} = \frac{(0.25)(2)}{1 + 0.75} \log\left(\frac{208.99 + 9.2}{208.99}\right) = 0.0054 \text{ m} = \mathbf{5.4\ mm}$$

Hence, the total settlement is

$$\Delta s_{c(g)} = 162.4 + 15.7 + 5.4 = \mathbf{183.5\ mm}$$