付録 A　講義及びセミナー資料

A-4　上部工の設計概論（鋼橋）

1

Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Superstructure of Steel Box Girder Design

14 Nov 2017

Design of Steel Box Girder

How to Fabricate Stiffened Plates into Box Shape?

1. Steel Material
2. Marking on Steel plate
3. Cutting Steel Plate
4. Welding pieces onto Plate
5. Assembling of Plates

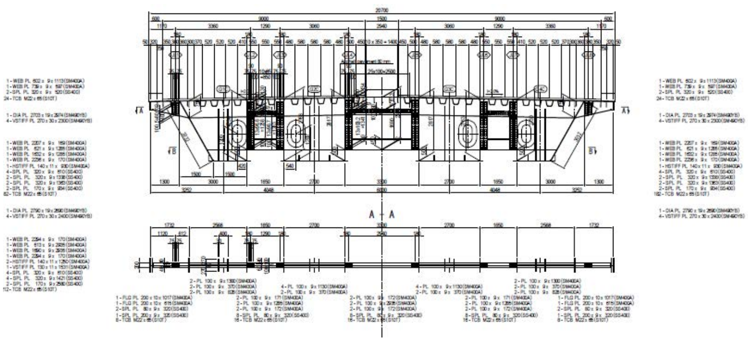
- Fabrication procedure depends on equipments that will be prepared by the fabricator.

From Shop Erection to Site Erection

6. Shop Erection
   - Using Multi staging not to deflect by self-steel weight.
   - Accuracy check of Dimension, Camber and Matching of Bolt Holes
7. Shop Painting
   - In accordance with Painting System
8. Site Erection using by Crawler Crane
   - Stress Check during whole Erection Stage
9. Completion of Girder Erection
10. Installation of Deck Slab

How to Prepare Steel Materials

1. Material Shown on The Design Drawings



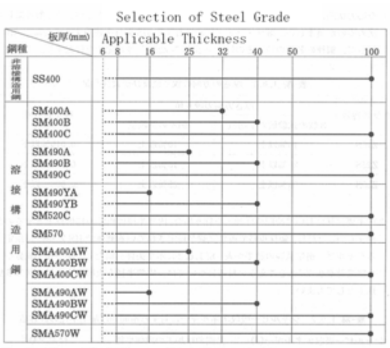
How to Prepare Steel Materials

２． Material List based on The Design

CP名称	原板番号	材質	表面処理	加工系列	板厚	幅	長さ	員数	ブロック	使用箇所	構造区分1
BA3	401	SM490YA-Z25	M33	P	14	2760	12411	1	G3B5	UFLG2	主桁
BA3	402	SM490YA-Z25	M33	P	14	2760	12479	1	G2B5	UFLG2	主桁
BA3	403	SM490YA-Z25	M33	P	14	2760	12479	1	G2B1	UFLG2	主桁
BA3	404	SM490YA-Z25	M33	P	14	2760	12411	1	G3B1	UFL	
BA3	405	SM490YB	M33	P	19	2757	11203	1	G3B4	UFL	
BA3	406	SM490YB	M33	P	19	2757	11257	1	G2B4	UFL	
BA3	407	SM490YB	M33	P	19	2757	11257	1	G2B2	UFL	
BA3	408	SM490YB	M33	P	19	340	340	1	G2B4	HAN	
BA3	408	SM490YB	M33	P	19	340	340	1	G2B2	HAN	
BA3	408	SM490YB	M33	P	19	340	340	1	G3B4	HAN	
BA3	408	SM490YB	M33	P	19	340	340	1	G3B2	HAN	
BA3	408	SM490YB	M33	P	19	2757	11203	1	G3B2	UFL	
BA3	409	SM490YB	M33	P	22	75	75	2	G1B5	DRL	
BA3	409	SM490YB	M33	P	22	75	75	2	G1B1	DRL	
BA3	409	SM490YB	M33	P	22	75	75	2	G2B5	DRL	
BA3	409	SM490YB	M33	P	22	75	75	2	G2B1	DRL	
BA3	409	SM490YB	M33	P	22	75	75	2	G3B5	DRL	
BA3	409	SM490YB	M33	P	22	75	75	2	G3B1	DRL	
BA3	409	SM490YB	M33	P	22	2753	9850	1	G2B3	UFL	
BA3	410	SM490YB	M33	P	22	340	340	1	G1B5	HAN	
BA3	410	SM490YB	M33	P	22	340	340	1	G1B1	HAN	
BA3	410	SM490YB	M33	P	22	2753	9802	1	G3B3	UFL	
BA3	411	SM490YB-Z25	M33	P	22	2761	12547	1	G1B5	UFL	
BA3	412	SM490YB-Z25	M33	P	22	2761	12547	1	G1B1	UFL	
BA3	413	SM490YB	M33	P	25	2757	11312	1	G1B4	UFL	
BA3	414	SM490YB	M33	P	25	340	340	1	G1B4	HAN	
BA3	414	SM490YB	M33	P	25	340	340	1	G1B2	HAN	
BA3	414	SM490YB	M33	P	25	2757	11312	1	G1B2	UFL	

Category	Grade	Thickness・Size	Main Girder	Deck Plate Including Cross Beam	Steel Materials for Accessaries	Sum
PL	SM490YB	38	11144			11,144
		28	22354			22,354
		27	7250	106,679		113,929
		26	20758			20,758
		25	622	108,905		109,527
		23	9818			9,818
		22	48480	177,907		226,387
		21	68282	58,863		127,145
		20	64988	99,016		164,004
		19	251174	84,778		335,952
		18	38448	1,048		39,496
		17	48770	442		49,212
	SM490YB	38--17	592,088	637,638		1,229,726
	SM490YA	16	145,840	1,575,725		1,721,565
		15	80740	2,528		83,268
		14	44320	4,608		48,928
		13	15310	1,917		17,227
		12	442488	3,950		446,438
		11	386256	1,156		387,412
		10	4002	21,842		25,844
		9	89622	354,896		444,518
	SM490YA	16--9	1,208,578	1,966,622	18,328	3,193,528

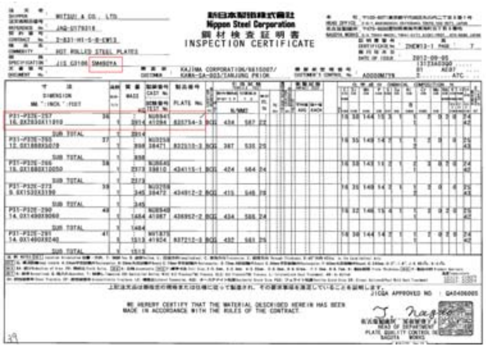
How to select the Steel Grade?



鋼種	Tensile Test Strength (N/mm²) 鋼材の厚さ(mm) 16以下	16をこえ40以下	40をこえ75以下	75をこえるもの	引張強さ (N/mm²)	Elongation 鋼材の厚さ(mm)	試験片*	伸び(%)	Charpy 記号	試験温度(℃)	シャルピー吸収エネルギー(J)
SS400	245以上	235以上	215以上	215以上	400〜510	16以下 16をこえ50以下 40をこえるもの	1A号 1A号 4 号	17以上 21以上 23以上	–	–	–
SM400	245以上	235以上	215以上	215以上	400〜510	16以下 16をこえ50以下 40をこえるもの	1A号 1A号 4 号	18以上 22以上 24以上	A B C	– 0 0	– 27以上 47以上
SMA 400W	245以上	235以上	215以上	215以上	400〜540	16以下 16をこえるもの 40をこえるもの	1A号 1A号 4 号	17以上 21以上 23以上	A B C	– 0 0	– 27以上 47以上
SM490	325以上	315以上	295以上	295以上	490〜610	16以下 16をこえ50以下 40をこえるもの	1A号 1A号 4 号	17以上 21以上 23以上	A B C	– 0 0	– 27以上 47以上
SM 490Y	365以上	355以上	335以上	325以上	490〜610	16以下 16をこえ50以下 40をこえるもの	1A号 1A号 4 号	15以上 19以上 21以上	A B	– 0	– 27以上
SMA 490W	365以上	355以上	335以上	325以上	490〜610	16以下 16をこえるもの 40をこえるもの	1A号 1A号 4 号	15以上 19以上 21以上	A B C	– 0 0	– 27以上 47以上
SM520	365以上	355以上	335以上	325以上	520〜640	16以下 16をこえ50以下 40をこえるもの	1A号 1A号 4 号	15以上 19以上 21以上	C	0	47以上
SM570	460以上	450以上	430以上	420以上	570〜720	16以下 16をこえるもの 20をこえるもの	5 号 5 号 4 号	19以上 26以上 20以上	–	-5	47以上
SMA 570W	460以上	450以上	430以上	420以上	570〜720	16以下 16をこえるもの 40をこえるもの	5 号 5 号 4 号	19以上 26以上 20以上	–	-5	47以上

How to Prepare Steel Materials

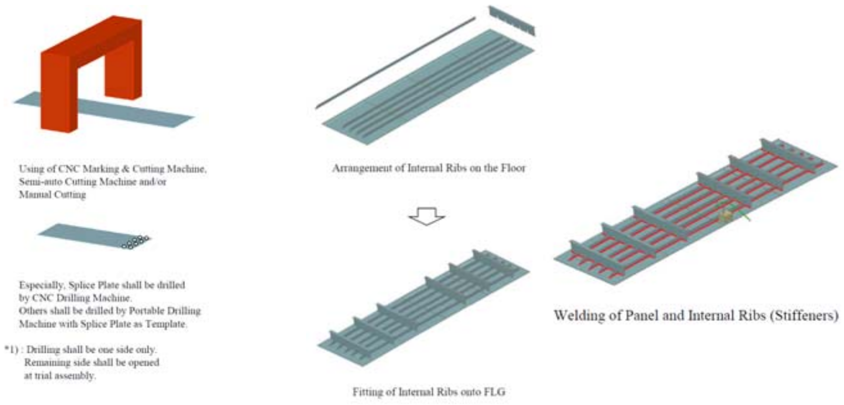
３． What is Mill Sheet？
（Certificate by Manufacturer）


MITSUI & CO., LTD.
HOT ROLLED STEEL PLATES
JIS G3106 SM490YA
新日本製鐵株式會社
Nippon Steel Corporation
鋼材検査証明書
INSPECTION CERTIFICATE
2012-09-05
KAJIMA CORPORATION/
WE HEREBY CERTIFY THAT THE MATERIAL DESCRIBED HEREIN HAS BEEN MADE IN ACCORDANCE WITH THE RULES OF THE CONTRACT.
HEAD OF DEPARTMENT
PLATE QUALITY CONTROL
NAGOYA WORKS

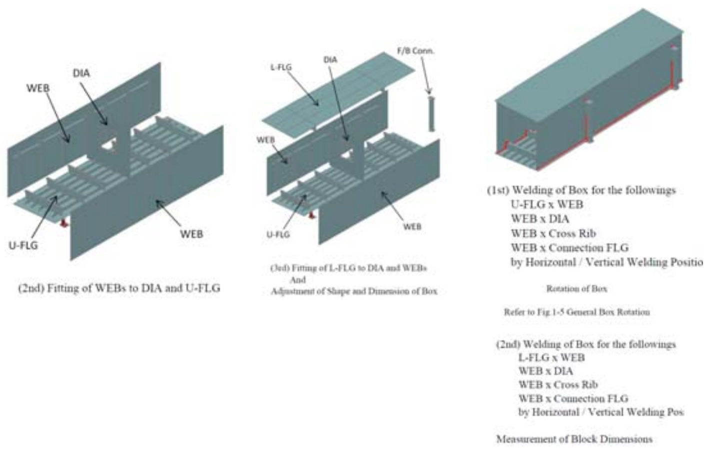
How to Prepare Steel Materials

４． Delivered Steel Plates
at the Storage Position

What is the procedure of assembling Box Shape?

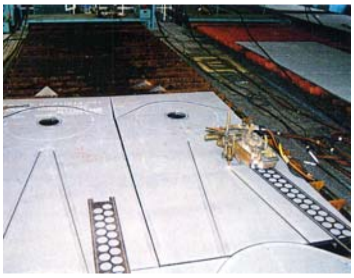


What procedure to assemble Box Shape by various plates is?

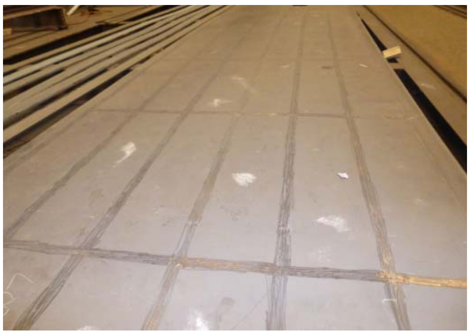


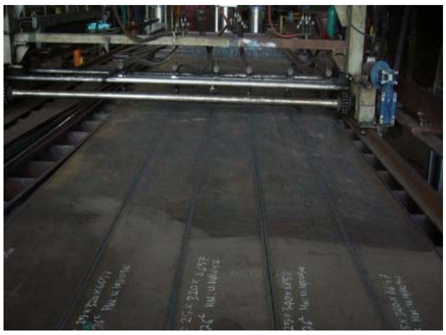
How to cut steel plates？

- Cutting Plan based on Shop Drawings and Piece Drawings.
- NC Marking GAS Frame Cut



What is Marking?





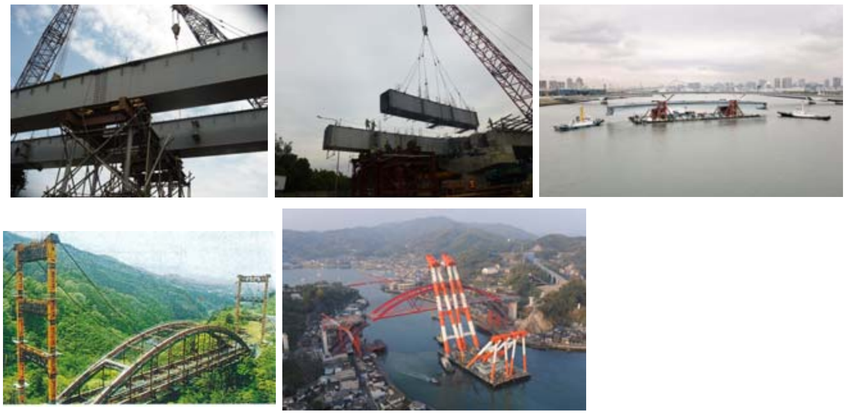
JICA Survey Team 13

JICA Survey Team 14

Traveling of Segment to Trial Erection Yard

What is the purpose of Trial Erection at the Shop?

There are several kinds of Erction Method.



Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Superstructure of Steel Box Girder Design

Nov 2017

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6. Several Kinds of Bridge Type
   - Girder Type
   - Truss Type
   - Arch Type
   - What Forces is primary on its designing?
7. Calculation Process for Bending Moment and Reaction of 2 span Girder.

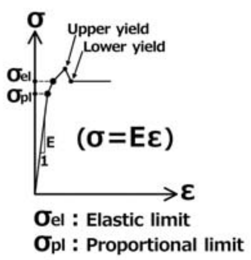
The Theoretical Assumption of Steel Bridge Design

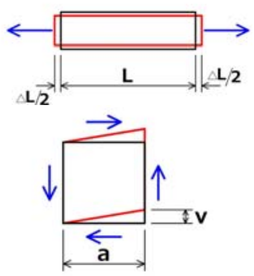
By Prf.Nagai's PPT

1. Hooke's Law →Bernoulli Principle

Stress-Strain is on linear relation

$\sigma = E\varepsilon \qquad \tau = G\gamma$





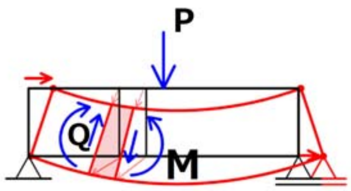
$$\varepsilon = \frac{\triangle L}{L}$$

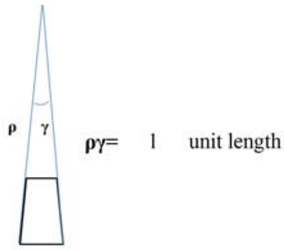
$$\gamma = \frac{v}{a}$$

The Theoretical Assumption of Steel Box Girder Design

2. **Bernoulli Assumption**

Deformed Section is keeping its plane as long as deformation(distortion) is not so large .
(The Section is always right angle to Neutral Axis)





The Theoretical Assumption of Steel Box Girder Design

3. **Equilibrium Relation of External/Internal force shall be sustained as long as the structure is stable.**

External Force(Load : Action)

↓

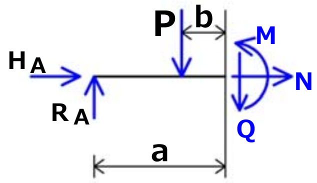
Internal Force(Bending Moment, Shearing Force, Torsional Moment)

↓

External Force(Reaction)

[Next step（Stress resultants N,M,Q）]

By Prf.Nagai's PPT



$N + H_A = 0 \rightarrow N = -H_A$

$Q + P = R_A \rightarrow Q = R_A - P$

$M + Pb - R_A a = 0 \rightarrow M = R_A a - Pb$

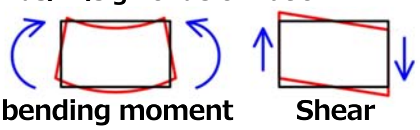
In equilibrium state

↓

Stress resultant
(Internal force)

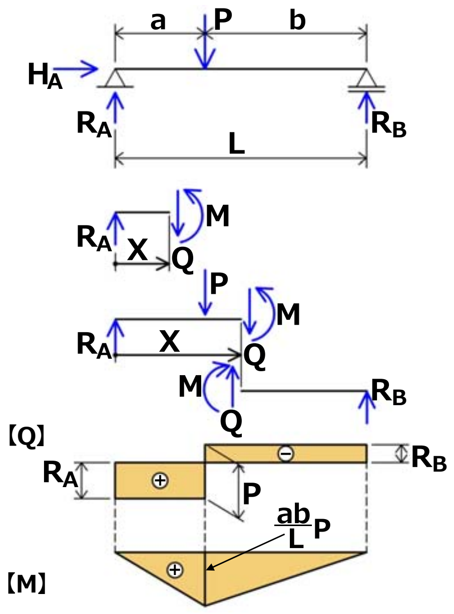
N,M,Q are obtained

Plus(+)sign of deformation



When（−）sign is obtained,
direction is reverse.

By Prf.Nagai's PPT



$H_A = 0$

$R_A L = Pb \rightarrow R_A = \frac{b}{L}P$

$R_B L = Pa \rightarrow R_B = \frac{a}{L}P$

$Q = R_A$

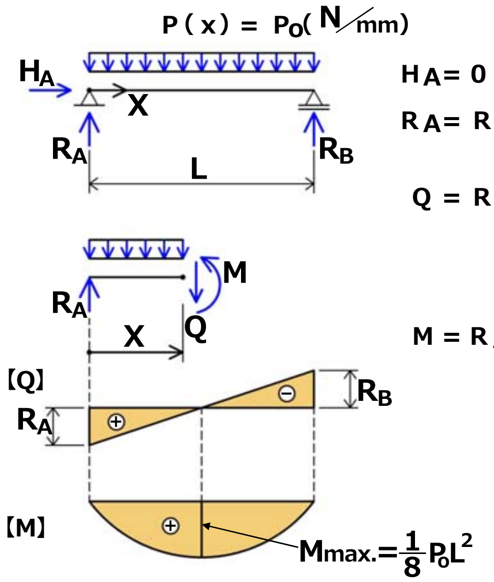
$M = R_A x$

$Q = R_A - P$

$M = R_A x - P(x - a)$

$(M = R_B(L - x))$

By Prf.Nagai's PPT



$$H_A = 0$$

$$R_A = R_B = \frac{P_o L}{2}$$

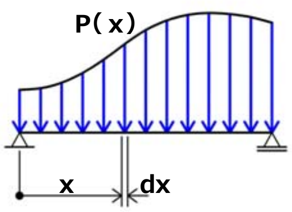
$$Q = R_A - P_o x = \frac{P_o L}{2} - P_o x$$

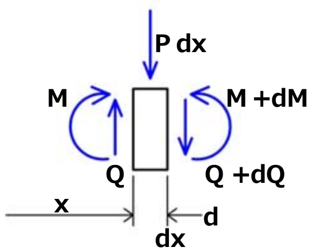
$$= P_o\left(\frac{L}{2} - x\right)$$

$$M = R_A x - (P_o x)\left(\frac{x}{2}\right) = \frac{P_o L}{2}x - \frac{P_o x^2}{2}$$

$$= \frac{P_o x}{2}(L - x)$$

By Prf.Nagai's PPT





$$Q + dQ + P(x)dx = Q$$

$$\frac{dQ}{dx} = -P(x)$$

$$M + (P(x)\,dx)\left(\frac{dx}{2}\right) + (Q + dQ)\,dx = M + dM$$

$$M + Q\,dx = M + dM$$

$$\boxed{\frac{dM}{dx} = Q}$$

$$\frac{d^2M}{dx^2} = -P(x)$$

JICA Survey Team 11

Equilibrium Equation of Tensile/Compression Force

Neutral Axis

$e(y) = \gamma \cdot y \quad \gamma$:constant ,

$\sigma(y) = E \cdot e(y)$

$= E \cdot \gamma \cdot y$

Equilibrium of Horizontal Force H:

$H = \int (\sigma \cdot dA)$

$= \int E \cdot \gamma \cdot y \, dA$

$= E \cdot \gamma \cdot \int y dA$

Herein $\int (ydA)$ is shown in case of I section.

$\int (ydA) = b1 \cdot \int ydy + b2 \cdot \int ydy + b3 \cdot \int ydy$

$= b1 \cdot (y1^2 - y2^2)/2 \quad \cdots\cdots$

$= b1 \cdot (y1-y2)(y1+y2)/2 \cdots\cdots$

$= b1 \cdot t1 \cdot y01 \cdots$

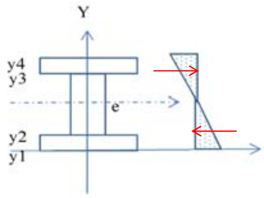
$= A1 \cdot y01 + A2 \cdot y02 + A3 \cdot y03$

$= A1 \cdot (y01' - e) + A2 \cdot (y02' - e) + A3 \cdot (y03' - e)$

$= A1 \cdot y01' + A2 \cdot y02' + A3 \cdot y03' - e \cdot (A1 + A2 + \cdots\cdots)$

$= 0$

$e = (A1 \cdot y01' + A2 \cdot y02' + A3 \cdot y03') / (A1 + A2 + \cdots\cdots)$



JICA Survey Team 12

Geometrical Moment of Inertia

$M = \int (\sigma \cdot dA \cdot y)$

$= \int E \cdot \gamma \cdot y^2 \cdot dA$

$dA = b \cdot dy$

b:const if shape is rectangular.

$M = E \cdot \gamma \cdot \int b \cdot y^2 dy$

$= E \cdot \gamma \cdot b \cdot \int y^2 dy$

If Y: Local coordinate of each section

$= E \cdot \gamma \cdot b \cdot \int (Y + y_0)^2 dy$

$= E \cdot \gamma \cdot b \cdot \int (Y + y_0)^2 dy$

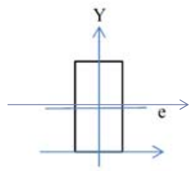
Herein

$b \cdot \int (Y + y_0)^2 dy = b \cdot \int (Y^2 + 2y_0 Y + yo^2) dy$

$= b \cdot \int Y^2 dy + b \cdot \int 2y_0 Y dy + b \cdot \int yo^2 dy$

$= I0 + 0 + yo^2 \cdot A$

$I0 = b/3 \cdot [Y^3]^{h/2}_{-h/2} = bh^3/12$



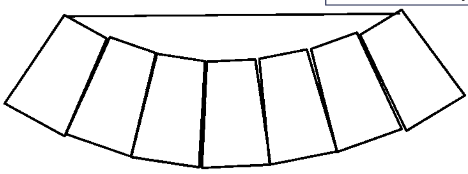
JICA Survey Team 13

Deflection due to Bending Moment

Radius $\rho=1/\gamma$
$1/\rho=\pm d^2y/dx^2$

$\sigma(y)=E\cdot e(y)$
$=E\cdot\gamma\cdot y$
$\gamma=M/EI$

$d^2y/dx^2=M/EI$



JICA Survey Team 14

Confirmation of Section Properties

1. Geometrical Moment of Area at **Optional Axis**

$$\int(y-y_0)dA=\int ydA-\int y_0dA$$
$$=G-y_0\cdot A$$
$$y_0=G/A \quad \text{:Distance to Neutral Axis}$$

2. Geometrical Moment of Inertia

$$y=Y+yo$$
$$\int y^2dA=\int Y^2dA+2y0\int YdA+yo^2\cdot A$$
$$=I0+\underline{2yo\cdot G}+y0^2\cdot A$$
$$=I0+y0^2\cdot A$$

Confirmation of Section Properties

3. Section Modulus

$Z= I/ y(max/min)$:Definition

$\gamma=M/(E \cdot I)$,

$\sigma(y)=E \cdot \gamma \cdot y$

$=y \cdot M/I$

$=M/Z$

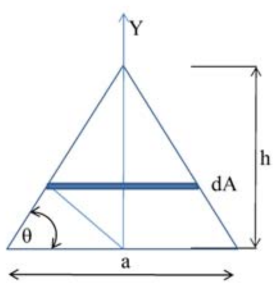
4. Radius of Gyration of Area

$r = \sqrt{(I/A)}$

Practice of Calculation 1

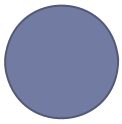
1. What is Geometrical Moment of Inertia ?

(Please Calculate by integration equation)



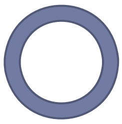
Practice of Calculation 2

1. What is Geometrical Moment of Inertia ?
2. What is Radius of Gyration of Area?



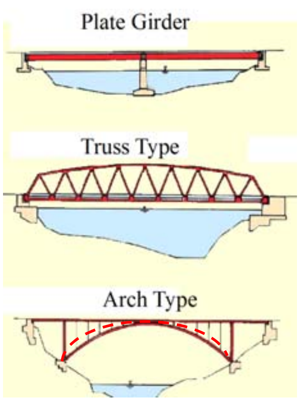
Radius of Circle is r

Half Circle Radius of Circle is r



Radius of outer Circle is r
Inner Circle is r-t

Different Design Forces related to Several Kinds of Bridge Type



Bending Moment & Shearing Force
$\sigma b = M/Z < \sigma a$, $\tau = S/Aw < \tau a$

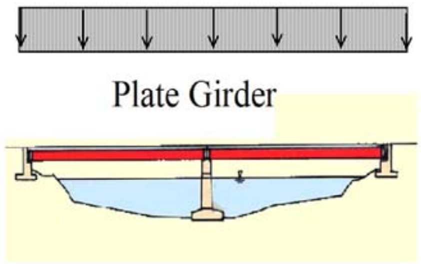
Axial Force
$\sigma = P/A < \sigma cr$ or σa (Chord & Diagonal)

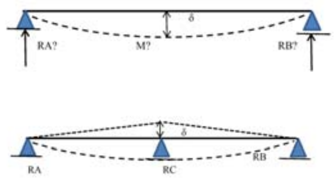
Axial Force & Bending Moment
$\sigma = \sigma p + \sigma b = P/A + M/Z < \sigma ca$

Calculation Process for Bending Moment & Reaction

Uniform Load w=28kn/m (nearly 3 Lanes AASHTO Loadin)

Total weight is 28*100= 2800kn





Calculation of B.M and Reaction 1

Process for Reaction of Mid span

RA=RB= w・L/2

M= wx/2・(L-x)

= w・L²/8 :(x= 0 to L/2)

EI・δ(y)= ∫∫(M)dx² =

= **w/2・∫∫(Lx-x²) dx2**

= **w/2・∫(Lx²/2-x³/3+C1) dx** dy/dx=0 at x=L/2

= **w/2・(Lx³/6-x⁴/12+C1x+C2)** y=0 at x=0

C1=-L³/12

C2=0

= **w/2・(L⁴/48- L⁴/192 - L⁴/24)**

= **5wL⁴/384**

JICA Survey Team 21

Calculation of B.M and Reaction 2

$$M = RA \cdot L/2 = RC \cdot L/4$$

$$\delta(y) = \int\int (M/EI)dx^2 =$$

$$EI \cdot \delta(y) = \int\int RC/2 \cdot x dx^2 =$$

$$= RC/2 \int\int x dx^2 =$$

$$= RC/2 \int (x^2/2 + C1) dx$$

at $x = L/2$, $dy/dx = 0$

$C1 = -L^2/8$

$$= RC/2 \cdot \{[x^3/6 + C1 \cdot x + C2\}$$

at $x = 0$, $y = 0$

$C2 = 0$

$$= RC/2 \cdot \{[x^3]/6 \ -L^2/8 \cdot x\}$$

$$= RC/2 \cdot \{L^3/48 \ -L^2/8 \cdot L/2\}$$

$$= RC \cdot L^3/48$$

$$5wL^4/384 - RC \cdot L^3/48 = 0$$

$$RC = 5 \cdot 48wL/384 = 5wL/8$$

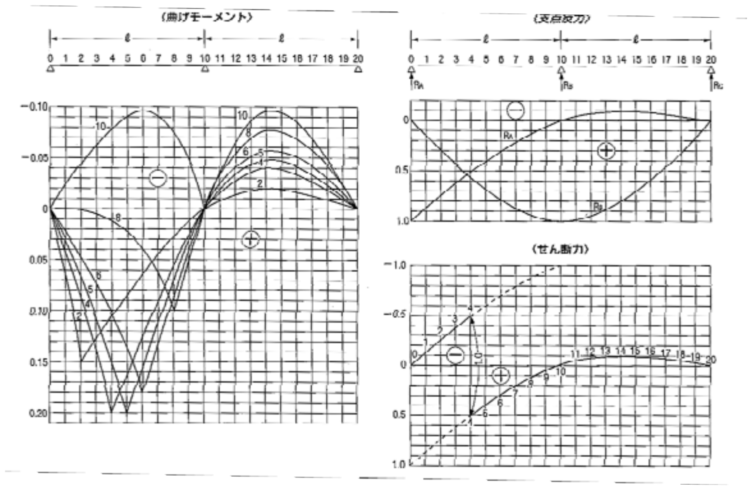
$$MC = w \cdot L^2/8 - RC \cdot L/4$$

$$= w \cdot L^2/8 - 5wL^2/32$$

$$= -wL^2/32$$

JICA Survey Team 22

Influence Line Diagram Method



JICA Survey Team 23

Influence Line Value and Area

載荷点	着目点													
	曲げモーメント(kN・m)										せん断力(kN)		支点反力(kN)	
	1	2	3	4	5	6	7	8	9	10	$\ell_1(Q_0)$	$\ell_2(Q_{10})$	R_A	R_B
0	0	0	0	0	0	0	0	0	0	0	1.0000	0	1.0000	0
1	0.0875	0.0751	0.0626	0.0501	0.0376	0.0252	0.0127	0.0002	-0.0123	-0.0248	0.8753	0.0248	0.8753	0.1495
2	0.0752	0.1504	0.1256	0.1008	0.0760	0.0512	0.0264	0.0016	-0.0232	-0.0480	0.7520	0.0480	0.7520	0.2960
3	0.0632	0.1264	0.1895	0.1527	0.1159	0.0791	0.0422	0.0054	-0.0314	-0.0683	0.6318	0.0683	0.6318	0.4365
4	0.0516	0.1032	0.1548	0.2064	0.1580	0.1096	0.0612	0.0128	-0.0356	-0.0840	0.5160	0.0840	0.5160	0.5680
5	0.0406	0.0812	0.1219	0.1625	0.2031	0.1438	0.0844	0.0250	-0.0344	-0.0938	0.4063	0.0938	0.4063	0.6875
6	0.0304	0.0608	0.0912	0.1216	0.1520	0.1824	0.1128	0.0432	-0.0264	-0.0960	0.3040	0.0960	0.3040	0.7920
7	0.0211	0.0422	0.0632	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0103	-0.0893	0.2108	0.0893	0.2108	0.8785
8	0.0128	0.0256	0.0384	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.0152	-0.0720	0.1280	[illegible]	[illegible]	[illegible]
9	0.0057	0.0115	0.0172	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0.0515	-0.0428	0.0573	[illegible]	[illegible]	[illegible]
10	0	0	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	0	0	[illegible]	[illegible]	[illegible]
11	-0.0043	-0.0086	-0.0128	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0385	-0.0428	-0.0428	[illegible]	[illegible]	[illegible]
12	-0.0072	-0.0144	-0.0216	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0648	-0.0720	-0.0720	[illegible]	[illegible]	[illegible]
13	-0.0089	-0.0179	-0.0268	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0803	-0.0893	-0.0893	[illegible]	[illegible]	[illegible]
14	-0.0096	-0.0192	-0.0288	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0864	-0.0960	-0.0960	[illegible]	[illegible]	[illegible]
15	-0.0094	-0.0188	-0.0281	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0844	-0.0938	-0.0938	[illegible]	[illegible]	[illegible]
16	-0.0084	-0.0168	-0.0252	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0756	-0.0840	-0.0840	[illegible]	[illegible]	[illegible]
17	-0.0068	-0.0137	-0.0205	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0614	-0.0683	-0.0683	[illegible]	[illegible]	[illegible]
18	-0.0048	-0.0096	-0.0144	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0432	-0.0480	-0.0480	[illegible]	[illegible]	[illegible]
19	-0.0025	-0.0050	-0.0074	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.0223	-0.0248	-0.0248	[illegible]	[illegible]	[illegible]
20	0	0	0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	0	0	0	[illegible]	[illegible]	[illegible]
A_1	0.0388	0.0675	0.0863	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	-0.01[illegible]3	-0.0625	0.4375	0.0625	0.4375	0.6250
A_2	-0.0063	-0.0125	-0.0188	-0.0250	-0.0313	-0.0375	-0.0438	-0.0500	-0.0563	-0.0625	0.0625	0.5625	-0.0625	0.6250
ΣA	0.0325	0.0550	0.0675	0.0700	0.0625	0.0450	0.0175	-0.0200	0.0675	-0.1250	0.3750	0.6250	0.3750	1.2500
					$\times \ell^2 (m^2)$								$\times \ell (m)$	

-0.0625
-0.0625
-0.1250

0.6250
0.6250
1.2500

JICA Survey Team 24

Trial Calculation of Section

- Design Bending Moment

Combined Mom(

$$MC = w \cdot L^2/8 - RC \cdot L/4$$
$$= w \cdot L^2/8 - 5wL^2/32$$
$$= -wL^2/32$$

- In case of L=100m, wl=28Kn/m
- Mc=-8759kn-m

  If Dead Load Moment is assumed same as Mc, total moment comes to [-8759x2=-17,518 kn-m]
- What dimension of box section is suitable?
- Please try!

Trial Calculation

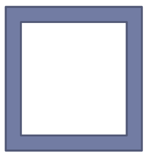
1. Required Z(Section Modulus)?

- In case that steel grade is SM490Y,

$\sigma a = 200N/mm^2 = 20Kn/cm^2$

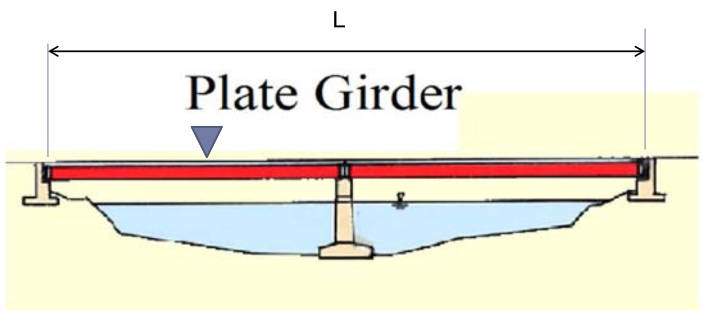
$Z > M/\sigma a$

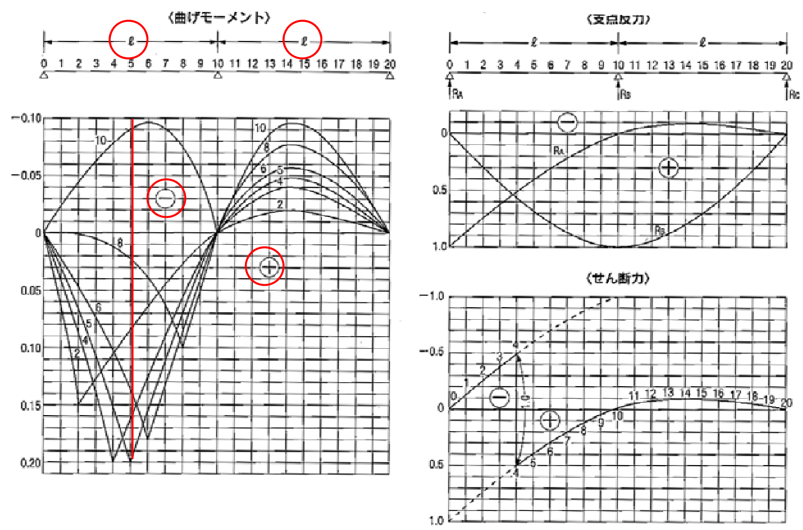
$> 1{,}751{,}800/20 = 87{,}590cm^3$



If you choose box section,
What height of web is?
What width of flange?
How thick is plate?
(including stiffener)

Practice Calculation 3
Bending Moment at ¼ span

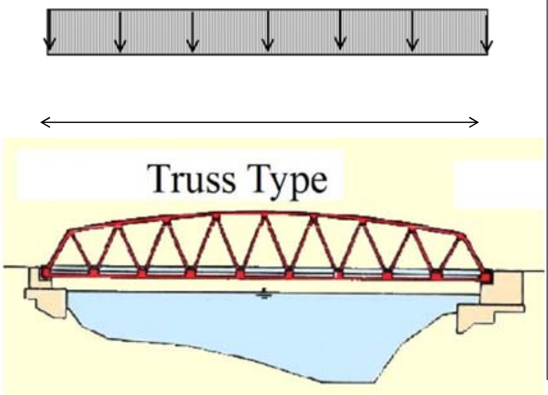




載荷点	着目点													
	曲げモーメント (kN・m)										せん断力(kN)		支点反力(kN)	
	1	2	3	4	5	6	7	8	9	10	ℓ₁(Q_B)	ℓ₂(Q_B)	R_A	R_B
0	0	0	0	0	0	0	0	0	0	0	1.0000	0	1.0000	0
1	0.0875	0.0751	0.0626	0.0501	0.0376	0.0252	0.0127	0.0002	-0.0123	-0.0248	0.8753	0.0248	0.8753	0.1495
2	0.0752	0.1504	0.1256	0.1008	0.0760	0.0512	0.0264	0.0016	-0.0232	-0.0480	0.7520	0.0480	0.7520	0.2960
3	0.0632	0.1264	0.1895	0.1527	0.1159	0.0791	0.0422	0.0054	-0.0314	-0.0683	0.6318	0.0683	0.6318	0.4365
4	0.0516	0.1032	0.1548	0.2064	0.1580	0.1096	0.0612	0.0128	-0.0356	-0.0840	0.5160	0.0840	0.5160	0.5680
5	0.0406	0.0812	0.1219	0.1625	0.2031	0.1438	0.0844	0.0250	-0.0344	-0.0938	0.4063	0.0938	0.4063	0.6875
6	0.0304	0.0608	0.0912	0.1216	0.1520	0.1824	0.1128	0.0432	-0.0264	-0.0960	0.3040	0.0960	0.3040	0.7920
7	0.0211	0.0422	0.0632	0.0843	0.1054	0.1265	0.1475	0.0686	-0.0103	-0.0893	0.2108	0.0893	0.2108	0.8785
8	0.0128	0.0256	0.0384	0.0512	0.0640	0.0768	0.0896	0.1024	0.0152	-0.0720	0.1280	0.0720	0.1280	0.9440
9	0.0057	0.0115	0.0172	0.0229	0.0286	0.0344	0.0401	0.0458	0.0515	-0.0428	0.0573	0.0428	0.0573	0.9855
10	0	0	0	0	0	0	0	0	0	0	0	1.0000	0	1.0000
11	-0.0043	-0.0086	-0.0128	-0.0171	-0.0214	-0.0257	-0.0299	-0.0342	-0.0385	-0.0428	-0.0428	0.9428	-0.0428	0.9855
12	-0.0072	-0.0144	-0.0216	-0.0288	-0.0360	-0.0432	-0.0504	-0.0576	-0.0648	-0.0720	-0.0720	0.8720	-0.0720	0.9440
13	-0.0089	-0.0179	-0.0268	-0.0357	-0.0446	-0.0536	-0.0625	-0.0714	-0.0803	-0.0893	-0.0893	0.7893	-0.0893	0.8785
14	-0.0096	-0.0192	-0.0288	-0.0384	-0.0480	-0.0576	-0.0672	-0.0768	-0.0864	-0.0960	-0.0960	0.6960	-0.0960	0.7920
15	-0.0094	-0.0188	-0.0281	-0.0375	-0.0469	-0.0563	-0.0656	-0.0750	-0.0844	-0.0938	-0.0938	0.5938	-0.0938	0.6875
16	-0.0084	-0.0168	-0.0252	-0.0336	-0.0420	-0.0504	-0.0588	-0.0672	-0.0756	-0.0840	-0.0840	0.4840	-0.0840	0.5680
17	-0.0068	-0.0137	-0.0205	-0.0273	-0.0341	-0.0410	-0.0478	-0.0546	-0.0614	-0.0683	-0.0683	0.3683	-0.0683	0.4365
18	-0.0048	-0.0096	-0.0144	-0.0192	-0.0240	-0.0288	-0.0336	-0.0384	-0.0432	-0.0480	-0.0480	0.2480	-0.0480	0.2960
19	-0.0025	-0.0050	-0.0074	-0.0099	-0.0124	-0.0149	-0.0173	-0.0198	-0.0223	-0.0248	-0.0248	0.1248	-0.0248	0.1495
20	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	× ℓ (m)										×1.0			
A₁	0.0388	0.0675	0.0863	0.0950	0.0938	0.0825	0.0613	0.0300	-0.0113	-0.0625	0.4375	0.0625	0.4375	0.6250
A₂	-0.0063	-0.0125	-0.0188	-0.0250	-0.0313	-0.0375	-0.0438	-0.0500	-0.0563	-0.0625	-0.0625	0.5625	-0.0625	0.6250
ΣA	0.0325	0.0550	0.0675	0.0700	0.0625	0.0450	0.0175	-0.0200	0.0675	-0.1250	0.3750	0.6250	0.3750	1.2500
	× ℓ²(m²)										× ℓ (m)			

For Dead Load : Full Span Load 0.625・$(L/2)^2w$
Live Load: Positive Zone Load 0.938・$(L/2)^2w$

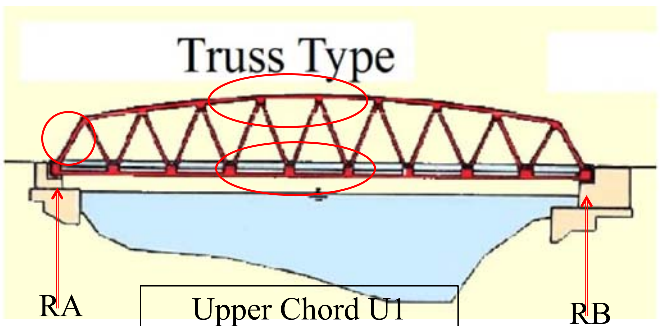
Calculation Process for Axial Force & Reaction



Loading at Lower Chord

Height of Truss h
Span Length L
Diagonal Inclination
60° (Assumption)

Practice Calculation of the red circled Member



Upper Chord U1
Lower Chord L1
End Diagonal D1

1

Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer

Lecture : Superstructure of Steel Box Girder Design

Dec 2017

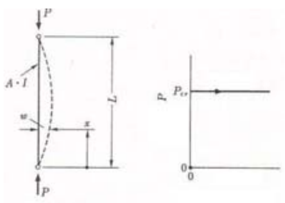
2

Contents

Contents

Buckling Stress 1

- Theoretical Equation
  Strain Energy due to Compression Stress
  ⇒ Strain Energy due to Bending Deformation
  (Theoretically, Buckling happens suddenly, but actual Buckling is occurred gradually because of existing initial deformation.)



Critical Load: P_{cr} Basic Buckling Stress: σ_{cr}

$\sigma_{cr}=P_{cr}/A$

Bending Moment: $M=P \cdot w$

Deflection due to M:w

$d^2w/dx^2=-M/EI=-Pw/EI$

$d^2w/dx^2 + k^2 \cdot w=0$

Buckling Stress 1

- The above equations shows the idealized relation.
- No eccentricity of member axis is assumed and also yield stress is higher than critical stress.
- Compression deformation moves to bending deformation when the stress reaches to critical stress σcr.
- On the above formula, P/EI is replaced by k^2.
- This is very skillful replacement.

Buckling Stress 2

- Basic Equation of Buckling and General Solution

$$d^2w/dx^2 + k^2 \cdot w = 0$$

$$w = \alpha \cdot \sin(kx) + \beta \cdot \cos(kx)$$

Taking account of boundary condition

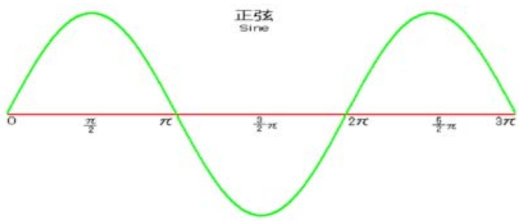
w=0 at x=0 and x=L

$$w = \alpha \cdot \sin(0) + \beta \cdot \cos(o) = \beta = 0$$

$$w = \alpha \cdot \sin(kL) = \alpha \cdot \sin(n \cdot \pi)$$

General Solution

- $\sin(kx)' = k \cdot \cos(kx)$, $\cos(kx)' = -k \cdot \sin(kx)$,
  $\sin(kx)'' = -k^2 \cdot \sin(kx)$, $\cos(kx)'' = -k^2 \cdot \cos(kx)$
  $d^2w/dx^2 = -k^2(\alpha\sin(kx)+\beta\cos(kx))$
  $= -k^2 w$



Relation between σcr and Radius of Gyration

$W = \alpha \cdot \sin(n \cdot \pi)$

$kL = n \cdot \pi$,

$k^2 = (n\pi/L)^2 = P/EI$

$P = n^2\pi^2 EI/L^2$,

$P_{cr} = \pi^2 EI/L^2$

$\sigma_{cr} = P_{cr}/A = \pi^2 EI/AL^2$

$= \pi^2 E/(L/r)^2$ $\quad (r = \sqrt{(I/A)})$

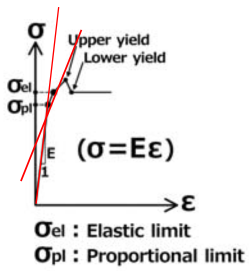
L/r: Slenderness Ratio

Relation between σ cr and Radius of Gyration

- This equation shows Buckling stress σcr is related to the ration of Column length and radius of gyration.
- It is expressed as Slenderness Ratio.
- According as length becomes shorter or radius becomes larger, σcr becomes larger.
- To increase (r), Moment of Inertia I should be larger even if area is same.

Comparison with Yield stress

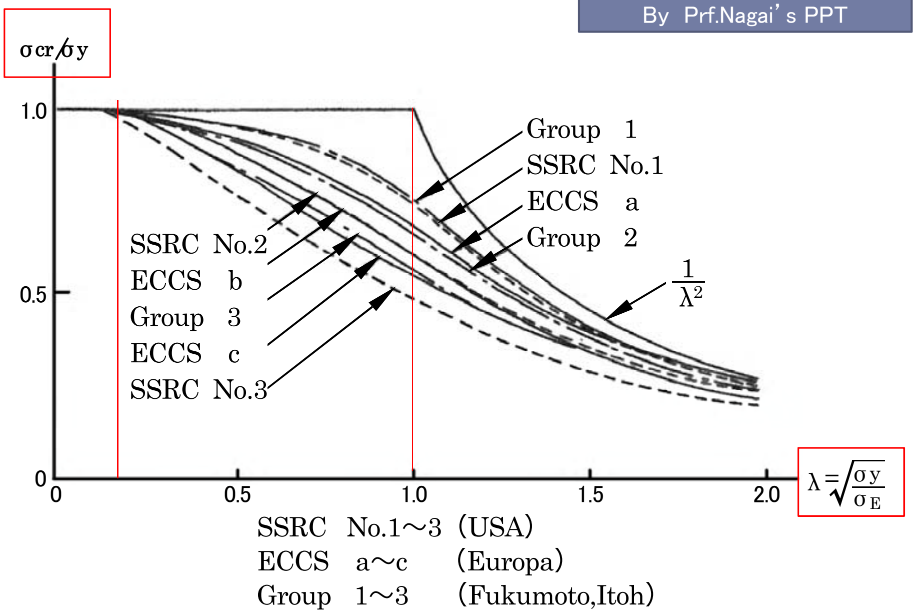
- $\sigma cri = Pcri/A = \pi^2 EI/AL^2 = \pi^2 Ee/(L/r)^2$ (Euler Elastic Buckling)
- $\sigma r = Pr/A = \pi^2 EI/AL^2 = \pi^2 Er/(L/r)^2 < \sigma y < \sigma cri$ (Buckling with initial deformation and deformation after yield)



σr is decided in accordance with the application formula based on experiment results.

Load carrying capacity curve for columns

By Prf.Nagai's PPT



Evaluation of σcr

- These Buckling Resistance Curve were derived from some experiments done by any other laboratories.
- Vertical Axis shows the ratio of σcr by σy, and Horizontal Axis shows the square root ratio of σy by the Euler's critical stress σe.

Japanese Standard of Cr

- $\lambda \leq 0.2$ $\sigma cr/\sigma y=1.0$
  $0.2 < \lambda \leq 1.0$ $\sigma cr/\sigma y=1.109-0.543\lambda$
  $1.0 < \lambda$ $\sigma cr/\sigma y=1.0/(0.773+\lambda^2)$

  $\lambda=\sqrt{\sigma y/\sigma e}$
  $\sigma e=\pi^2 E/(L/r)^2$
- $\sigma cr/(\text{safety factor}: 1.7)=\sigma ca$

- Buckling Resistance Curve is separated to 3 parts.
- σy is depend on material grade, and
- σe is depend on the ratio of gyration radius r and
- member length L.

Allowable stress of compression member without local buckling

鋼種 / 板厚(mm)	SS400 SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
40以下	$140: \frac{l}{r} \leq 18$ $140-0.82\left(\frac{l}{r}-18\right): 18 < \frac{l}{r} \leq 92$ $\frac{1,200,000}{6,700+\left(\frac{l}{r}\right)^2}: 92 < \frac{l}{r}$	$185: \frac{l}{r} \leq 16$ $185-1.2\left(\frac{l}{r}-16\right): 16 < \frac{l}{r} \leq 79$ $\frac{1,200,000}{5,000+\left(\frac{l}{r}\right)^2}: 79 < \frac{l}{r}$	$210: \frac{l}{r} \leq 15$ $210-1.5\left(\frac{l}{r}-15\right): 15 < \frac{l}{r} \leq 75$ $\frac{1,200,000}{4,400+\left(\frac{l}{r}\right)^2}: 75 < \frac{l}{r}$	$255: \frac{l}{r} \leq 18$ $255-2.1\left(\frac{l}{r}-18\right): 18 < \frac{l}{r} \leq 67$ $\frac{1,200,000}{3,500+\left(\frac{l}{r}\right)^2}: 67 < \frac{l}{r}$

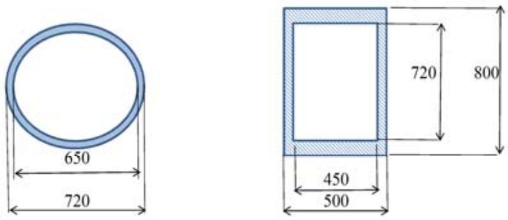
●This is the table of allowable stress relating to buckling of column.
●So, local buckling of plate is not considered at this moment.

Calculation Exercise

What is the allowable stress of each section?

Column Length L=5000mm, Steel material grade SM490Y

Do=	720	mm		Ho=	800	Hi=	720
Di=	650	mm		Wo=	500	Wi=	450

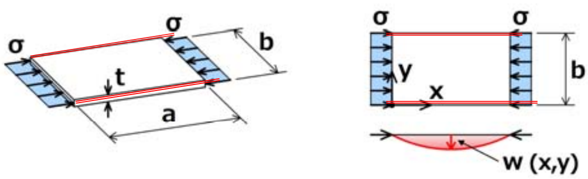


By Prf.Nagai' s PPT

Buckling of Un-Stiffend Plate

4 Edges are simply supported

Fundamental equation



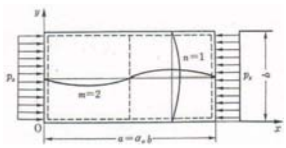
$$B\left(\frac{\partial^4 w}{\partial x^4} + 2\frac{\partial^4 w}{\partial x^2 \partial y^2} + \frac{\partial^4 w}{\partial y^4}\right) + (\sigma t)\frac{\partial^2 w}{\partial x^2} = 0 \quad \text{——(1)}$$

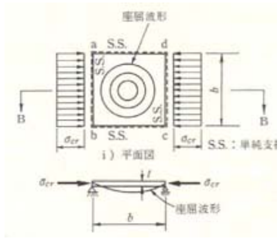
$$B = \frac{Et^3}{12(1-\nu^2)}$$

●Factor B shows bending stiffness of unit width panel considering about poisson ratio.

Buckling of Un-Stiffend Plate

- Plate deflection is 2dimensional. Usually, panel length a is larger than width b.





- Deflection figure of rectangular is like as contour line.

By Prf.Nagai' s PPT

Assuming, $w(x,y) = A_{m,n} \sin \frac{m\pi}{a} x \cdot \sin \frac{n\pi}{b} y$ ——(2)

(m , n = 1,2,3 ……)

satisfying boundary condition

$$\sigma_E = \frac{\pi^2 B}{b^2 t}\left(m\frac{b}{a} + \frac{n^2}{m}\frac{a}{b}\right)^2$$

min. value is obtained when n =1 , and setting $\alpha = a/b$ (α: aspect ratio)

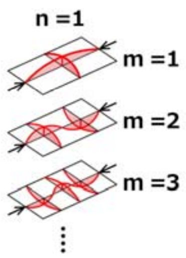
$$\sigma_E = \frac{\pi^2 B}{b^2 t}\left(\frac{m}{\alpha} + \frac{\alpha}{m}\right)^2$$

We rewrite

$$\sigma_E = k\underbrace{\left[\frac{\pi^2 E}{12(1-\nu^2)}\left(\frac{t}{b}\right)^2\right]}_{=\sigma_{E0}} = k \cdot \sigma_{E0}$$

$$k = \left(\frac{m}{\alpha} + \frac{\alpha}{m}\right)^2 \quad (m = 1,2,3 \cdots\cdots)$$

k : buckling coefficient of plate

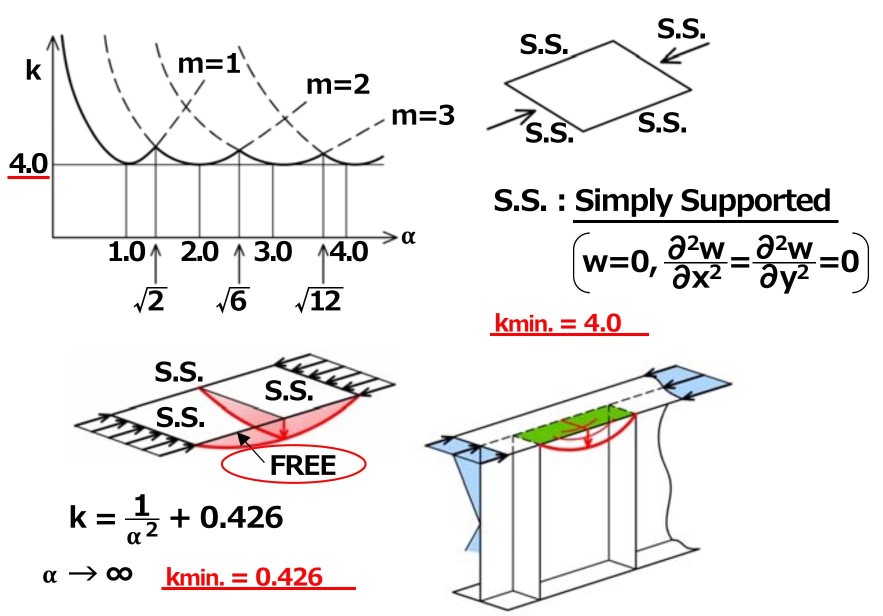


Buckling Coefficient

- Considering boundary condition,
- the deflection is 0 at for edges.
- $P_{cr} = B\pi^2 \{(m/a)^2 + (n/b)^2\}^2 / (m/a)^2$
- $\sigma_{cr} = B\pi^2/bt \cdot \{(m/a)^2 + (n/b)^2\}^2 / (m/a)^2$
- $= B\pi^2/bt \cdot \{(m/a) + n^2a/mb^2\}^2$
- $= B\pi^2/b^2t \cdot \{mb/a + n^2a/mb\}^2$
- $= B\pi^2/b^2t \cdot \{m/\alpha + n^2\alpha/m\}^2$
- $= B\pi^2/b^2t \cdot \{m/\alpha + \alpha/m\}^2$ (n=1)
- $= \pi^2 E/12(1-\nu^2) \cdot (t/b)^2 \cdot \{m/\alpha + \alpha/m\}^2$
- $= k \cdot \sigma_e$
- k : Buckling coefficient $\alpha = a/b$
- σ_e: Critical buckling stress without stiffeners
- If m=1, α=1 (a=b) k=4

10-2-3 図-④

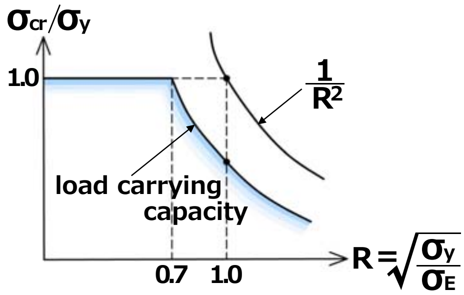
By Prf.Nagai’s PPT



Ultimate strength of unstiffened plate by JHBS

By Prf.Nagai's PPT

$$R = \sqrt{\frac{\sigma_y}{\sigma_E}} = \sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\nu^2)}{\pi^2 k}} \cdot \left(\frac{b}{t}\right)$$



$\frac{\sigma_{cr}}{\sigma_y} = 1.0$	$R \leqq 0.7$
$\frac{\sigma_{cr}}{\sigma_y} = 0.5/R^2$	$0.7 < R$

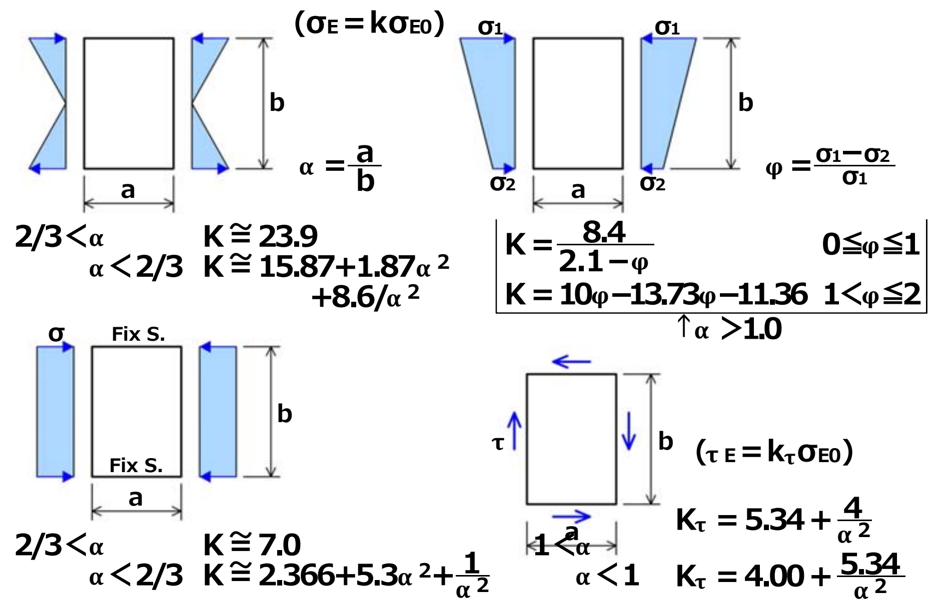
JICA Survey Team 22

The Relation between σe and σcr

- The above figure shows σcr is decreased in accordance with (b/t).
- This is based on the experimental results including initial deformation and residual stresses due to welding.
- $\sigma e = \pi^2 Ek/12(1-\nu^2)$
  This equation shows Larger k makes larger σe.
- Buckling Coefficient k is changed in accordance with stress distribution state.

K value depending on stress condition

By Prf.Nagai' s PPT



JICA Survey Team 24

Allowable stress is specified by b,t on JHBS (Simply support at 4-Edges)

SS400 SM400 SMA400W	40以下	140	$: \frac{b}{38.7f} \leqq t$
		$210,000\left(\frac{tf}{b}\right)^2$	$: \frac{b}{80f} \leqq t < \frac{b}{38.7f}$
	40を超え 100以下	125	$: \frac{b}{41.0f} \leqq t$
		$210,000\left(\frac{tf}{b}\right)^2$	$: \frac{b}{80f} \leqq t < \frac{b}{41.0f}$

SM490Y SM520 SMA490W	40以下	210	$: \frac{b}{31.6f} \leqq t$
		$210,000\left(\frac{tf}{b}\right)^2$	$: \frac{b}{80f} \leqq t < \frac{b}{31.6f}$
	40を超え 75以下	195	$: \frac{b}{32.8f} \leqq t$
		$210,000\left(\frac{tf}{b}\right)^2$	$: \frac{b}{80f} \leqq t < \frac{b}{32.8f}$

This table is shown as example, and f shows the stress gradient

Buckling of Stiffened Plate

Plate like as Flange and Web of Box Girder should be stiffened by many stiffeners. This procedure is most important on designing of steel box girder.

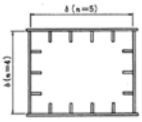


図-4.2.4 補剛板の全幅

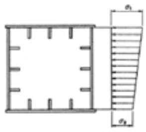


図-4.2.5 補剛板の縁応力度

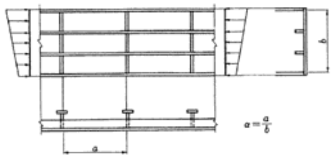
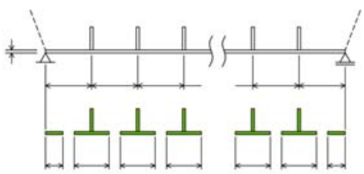


図-4.2.6 補剛板の縦横寸法比 α

Buckling of Stiffened Plate

- If considering about small panel that has width b and thickness t, and moreover with free side edge,
  The fundamental bending stiffness is
  $$EI=Ebt^3/12(1-\nu^2).$$
- Then, deflection of this panel is restricted by both web and stiffeners.
- The small part of stiffeners are assumed as column.

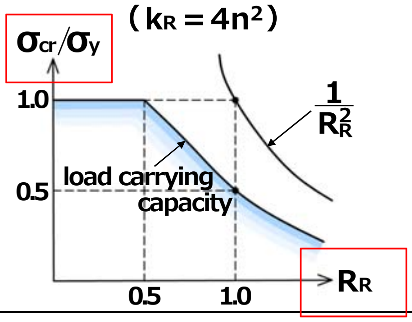


Ultimate strength of stiffened plate by JHBS

By Prf.Nagai's PPT

$$R_R = \sqrt{\frac{\sigma_y}{\sigma_E}} = \sqrt{\frac{\sigma_y}{E} \cdot \frac{12(1-\nu^2)}{\pi^2 k_R}} \cdot \left(\frac{b}{t}\right)$$

$(k_R = 4n^2)$



$\frac{\sigma_{cr}}{\sigma_y} = 1.0$	$R_R \leqq 0.5$
$\frac{\sigma_{cr}}{\sigma_y} = 1.5 - R_R$	$0.5 < R_R \leqq 1.0$
$\frac{\sigma_{cr}}{\sigma_y} = \frac{0.5}{R_R^2}$	$1.0 < R_R$

Buckling Coefficient k_R, k_F of Stiffened Plate and Stiffeners

- R_R=sqrt{(σy/E)・12(1-ν²)/π²k_R} (Ordinary Formula)
- k_R = Buckling Coefficient (=4n²)
  n:Panel numbers separated by stiffeners
- $k_F=\{(1+\alpha^2)^2+n\gamma_L\}/\{\alpha^2(1+n\delta_L)\}$ $(\alpha \leq \alpha_0)$
  $k_F=2\{1+\text{sqrt}(1+n\gamma_L)\}/(1+n\delta_L)$ $(\alpha > \alpha_0)$
  $\gamma_L=I_L/(bt^3/11)$:Stiffness Ratio of Stiffener
  $\delta_L=A_t/bt$:Area ratio of stiffener
- The above formula is based on the experimental results and theoretical equation.

Allowable Stress of Stiffened Plate against Local Buckling 1

鋼種 / 板厚 (mm)	SS400 SM400 SMA400W	SM490	SM490Y SM520 SMA490W	SM570 SMA570W
40以下	$140: \frac{b}{28fn} \leqq t$	$185: \frac{b}{24fn} \leqq t$	$210: \frac{b}{22fn} \leqq t$	$255: \frac{b}{22fn} \leqq t$
	$140 - 2.6\left(\frac{b}{tfn} - 28\right): \frac{b}{56fn} \leqq t < \frac{b}{28fn}$	$185 - 3.9\left(\frac{b}{tfn} - 24\right): \frac{b}{48fn} \leqq t < \frac{b}{24fn}$	$210 - 4.6\left(\frac{b}{tfn} - 22\right): \frac{b}{46fn} \leqq t < \frac{b}{22fn}$	$255 - 6.9\left(\frac{b}{tfn} - 22\right): \frac{b}{40fn} \leqq t < \frac{b}{22fn}$
	$210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{56fn}$	$210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{48fn}$	$210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{46fn}$	$210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{40fn}$

Allowable Stress of Stiffened Plate against Local Buckling 2

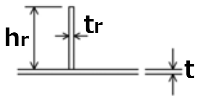
40を超え75以下	$125: \frac{b}{28fn} \leqq t$ $125 - 2.1\left(\frac{b}{tfn} - 28\right): \frac{b}{58fn} \leqq t < \frac{b}{28fn}$ $210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{58fn}$	$175: \frac{b}{24fn} \leqq t$ $175 - 3.5\left(\frac{b}{tfn} - 24\right): \frac{b}{50fn} \leqq t < \frac{b}{24fn}$ $210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{50fn}$	$195: \frac{b}{22fn} \leqq t$ $195 - 4.0\left(\frac{b}{tfn} - 22\right): \frac{b}{46fn} \leqq t < \frac{b}{22fn}$ $210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{46fn}$	$245: \frac{b}{22fn} \leqq t$ $245 - 6.2\left(\frac{b}{tfn} - 22\right): \frac{b}{42fn} \leqq t < \frac{b}{22fn}$ $210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{42fn}$
75を超え100以下			$190: \frac{b}{22fn} \leqq t$ $190 - 3.7\left(\frac{b}{tfn} - 22\right): \frac{b}{48fn} \leqq t < \frac{b}{22fn}$ $210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{48fn}$	$240: \frac{b}{22fn} \leqq t$ $240 - 6.0\left(\frac{b}{tfn} - 22\right): \frac{b}{42fn} \leqq t < \frac{b}{22fn}$ $210,000\left(\frac{tfn}{b}\right)^2: \frac{b}{80fn} \leqq t < \frac{b}{42fn}$

Design of longitudinal ribs (I_ℓ)

By Prf.Nagai's PPT

$$\gamma_\ell = \frac{I_\ell}{\frac{bt^3}{12(1-\nu^2)}} \cong \frac{I_\ell}{\frac{bt^3}{11}} \longrightarrow I_\ell \geq \frac{bt^3}{11} \cdot \gamma_{\ell,req.}$$

$$A_\ell = \frac{bt}{10n}$$

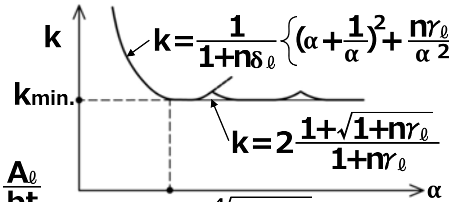


$$A_\ell = h_r t_r \qquad I_\ell = \frac{h_r^3 t_r}{3}$$

From condition,

$\sigma_E^{①}$ (buckling stress of plate between ribs) $=$ $\sigma_E^{②}$ (buckling stress of stiffened plate)

$$k = 4n^2$$



$$k = \frac{1}{1+n\delta_\ell}\left\{\left(\alpha+\frac{1}{\alpha}\right)^2 + \frac{n\gamma_\ell}{\alpha^2}\right\}$$

$$k = 2\frac{1+\sqrt{1+n\gamma_\ell}}{1+n\gamma_\ell}$$

$$\delta_\ell = \frac{A_\ell}{bt} \qquad \alpha_0 = \sqrt[4]{1+n\gamma_\ell}$$

$\alpha < \alpha_0$ | $\alpha > \alpha_0$

By Prf.Nagai's PPT

1) $\alpha \leqq \alpha_0$ (&) $I_t \geqq \frac{bt^3}{11} \cdot \frac{1+n\gamma_{\ell,req.}}{4\alpha^3}$

$$\gamma_{\ell,req.} = 4\alpha^2 n\left(\frac{t_0}{t}\right)^2(1+n\delta_\ell) - \frac{(\alpha^2+1)^2}{n} \qquad (t \geqq t_0) \quad (R_R < 0.5)$$

$$= 4\alpha^2 n(1+n\delta_\ell) - \frac{(\alpha^2+1)}{n} \qquad (t < t_0) \quad (R_R > 0.5)$$

(t_0 is the thickness when $R_R = 0.5$)

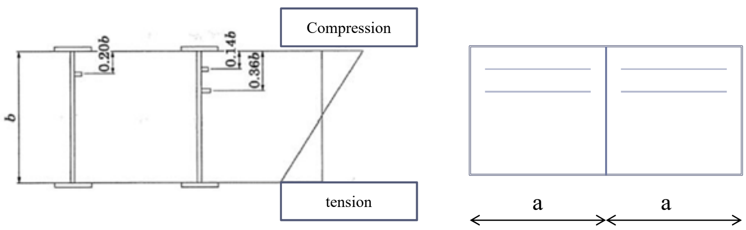
2) the others [$(\alpha > \alpha_0)$, $(\alpha \leqq \alpha_0$ & $I_t < \frac{bt^3}{11} \cdot \frac{1+n\gamma_{\ell,reg.}}{4\alpha^3})$]

$$\gamma_{\ell,reg.} = \frac{1}{n}\left[\left\{2n^2\left(\frac{t_0}{t}\right)(1+n\delta_\ell) - 1\right\}^2 - 1\right] \qquad (t \geqq t_0) \quad (R_R < 0.5)$$

$$= \frac{1}{n}\left[\left\{2n^2(1+n\delta_\ell) - 1\right\}^2 - 1\right] \qquad (t > t_0) \quad (R_R > 0.5)$$

Horizontal stiffener of web

- Web plate has stress gradient.



At Compression zone, Horizontal stiffener is required.

Design of Horizontal Stiffener 1

- Stress state should be satisfied with the following formula.

In case of no stiffener

$$\left(\frac{b}{100t}\right)^4\left[\left(\frac{\sigma}{345}\right)^2+\left\{\frac{\tau}{77+58(b/a)^2}\right\}^2\right]\leq 1:\left(\frac{a}{b}>1\right)$$

$$\left(\frac{b}{100t}\right)^4\left[\left(\frac{\sigma}{345}\right)^2+\left\{\frac{\tau}{58+77(b/a)^2}\right\}^2\right]\leq 1:\left(\frac{a}{b}\leq 1\right)$$

In case of single stiffener

$$\left(\frac{b}{100t}\right)^4\left[\left(\frac{\sigma}{900}\right)^2+\left\{\frac{\tau}{120+58(b/a)^2}\right\}^2\right]\leq 1:\left(\frac{a}{b}>0.80\right)$$

$$\left(\frac{b}{100t}\right)^4\left[\left(\frac{\sigma}{900}\right)^2+\left\{\frac{\tau}{90+77(b/a)^2}\right\}^2\right]\leq 1:\left(\frac{a}{b}\leq 0.80\right)$$

In case of dual stiffeners

$$\left(\frac{b}{100t}\right)^4\left[\left(\frac{\sigma}{3,000}\right)^2+\left\{\frac{\tau}{187+58(b/a)^2}\right\}^2\right]\leq 1:\left(\frac{a}{b}>0.64\right)$$

$$\left(\frac{b}{100t}\right)^4\left[\left(\frac{\sigma}{3,000}\right)^2+\left\{\frac{\tau}{140+77(b/a)^2}\right\}^2\right]\leq 1:\left(\frac{a}{b}\leq 0.64\right)$$

Design of Horizontal Stiffener 2

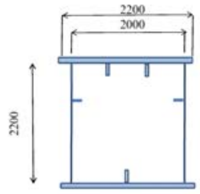
- Stiffenes of Horizontal Stiffener should be satisfied with the following formula.
- $I_h \geq bt^3/11 \cdot \gamma_{req}$
- $\gamma_{req}=30 \cdot (a/b)$
- Vertical stiffeners that separates web by a spacing is required it stiffeness more than $\gamma_{v.req}=8 \cdot (b/a)^2$

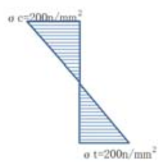
Calculation Exercise

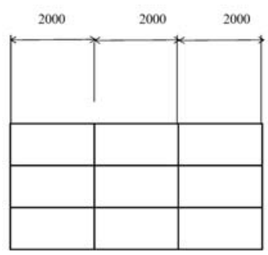
Allowable Stress of Flange and Dimension of Stiffeners

Design Condition

Width of Flange	W=	2200 mm
Distance of both webs	w=	2000 mm
Thickness of Flang	tf=	16 mm
Space of Cross beam	a=	2000 mm
Thickness of Web plate	tw=	9 mm
Sheraing Force of girder	S=	1980 Kn







付録 A　　　講義及びセミナー資料

A-5　　基礎工・下部工の設計概論

Detailed Design on Bago River Bridge Construction Project

JICA Study Team

Design Technology Transfer
Lecture : Substructure and Foundation Design

25th Oct 2017: Group B
26th Oct 2017: Group A

Introduction

- Substructure and Foundation Design -

1. LECTURE & BASIC PRACTICE STAGE

Objectives of the lecture

To study fundamental issues for Cast-In-Placed pile (CIP Pile) and Steel Pipe Sheet Pile (SPSP) with Reinforced Concrete Pier (RC Pier), which have been applied in New Thaketa Bridge Project and Bago River Bridge Project, through lectures and to learn necessary knowledges as a bridge engineer through exercises.

Contents of the lecture

Month	Program	Content	Lecturer
October	Substructure Design 1	Pre-Examination Substructure and Foundation Planning in General and Bago River Bridge Case	By Imada
November	Substructure Design 2	Design of CIP Pile (1), Exercise for CIP Pile	By Takaoka
November	Substructure Design 3	Design of CIP Pile (2), Exercise for CIP Pile Construction Methodology of SPSP Design of SPSP (1), Exercise for SPSP	By Takaoka, Imada
November	Substructure Design 4	Design of SPSP (2) , Exercise for SPSP	By Imada
December	Substructure Design 5	Design of RC Pier, Exercise for RC Pier	By Imada
December	Substructure Design 6	Mid-Term Examination	By Takaoka

Introduction

- Substructure and Foundation Design -

1. LECTURE & BASIC PRACTICE STAGE

Substructure Design 1
Among the various types of foundation, spread foundation, caisson foundation, pile foundation, steel pipe sheet pile foundation, diaphragm wall foundation and deep foundation will be briefly introduced, and important issues for selection of foundation type will be explained.
The reason why pile foundation (CIP pile) and SPSP were selected in Bago River Bridge will be explained with explanation of topographical, geological and environmental conditions.
Substructure Design 2
Design of CIP pile including setting design conditions, structural stability, sectional force, stress verification of piles, connection between pile and footing will be explained and some exercise will be done to deepen trainee's understanding.
Substructure Design 3 and 4
Lecture on the design of SPSP foundation will be held at two times. Firstly, to understand SPSP foundation, construction methodology of SPSP in New Thaketa Bridge Construction Project will be explained. Then, design of SPSP including setting design conditions, structural stability, stress verification of piles, temporary cofferdam design, connection between pile and footing will be explained and some exercise will be done to deepen trainee's understanding.
Substructure Design 5
Design of RC Pier including verifications at bottom of pier column, beam and bridge seat will be explained and some exercise will be done to deepen trainee's understanding.
Substructure Design 6
To check the trainees understanding on the contents of above five times lectures, mid-term examination (about two hours) will be implemented.

Introduction

- Substructure and Foundation Design -

2. DETAILED PRACTICE STAGE

<u>Objectives of the detailed practice</u>

To learn the design flow for SPSP, CIP Pile Foundations and RC Pier by using structural design software (Forum8) as a specific design exercise through 8 weeks.

The design method will apply allowable stress method including seismic design by static analysis against Level-1 earthquake in accordance with Japanese Specifications for Highway Bridge (JSHB).

<u>Contents of the lecture</u>

Week	Content	Lecturer
1st week 2nd week 3rd week	CIP Pile Design (modeling, setting design conditions, structural stability analysis, stress verification of piles, connection between pile and footing)	By Takaoka
4th week 5th week 6th week	SPSP Design (modeling, setting design conditions, structural stability analysis, temporary cofferdam design, stress verification of piles, connection between pile and footing)	By Imada
7th week 8th week	RC Pier Design (modeling, setting design conditions, verifications at bottom of pier column and beam, guideline of rebar arrangement)	By Imada

In the last week, based on the theory, knowledge and design method studied through the technical transfer program, design of substructure will be tried by trainee from the beginning to the end, and a presentation for result and summary of the technical transfer program is made by trainee as a conclusion.

Contents of Lecture (1)

- Substructure and Foundation Planning -

1. Foundation Planning in General
2. Substructure Planning in General
3. Plan of Substructure and Foundation for Bago River Bridge
4. Pre-Test

1. Foundation Planning in General

1.1 General concept for foundation planning

- Foundation of the bridge shall be planned accordance with overall structural planning, adjacent structures and topographical, geological, hydrological conditions, and environmental condition.
- The foundation planning shall be taken account for the construction easiness, reliability and cost effectiveness.
- Generally, the foundation type is considered after structural type and approximate size of super-structure is determined.

1. Foundation Planning in General

1.2 Pre-conditions for foundation planning

- In premise of foundation planning, each surveys to determine pre-condition shall be conducted, especially to confirm following conditions;
  - i) Topographical condition
  - ii) Geological condition
  - iii) Hydrological condition
  - iv) Environmental condition
    - - noise, vibration, pollution
    - - existing structure at neighbor
    - - restriction of machine use
    - - river condition

1. Foundation Planning in General

1.3 Type and feature of ground

a) Sandy ground : sandy ground is expected large bearing capacity which is determined mainly from internal friction angle (φ) and small settlement in general. Hence, it is utilized for bearing layer widely. However sandy ground having low N-value has less bearing capacity and possibility of liquefaction due to earthquake.

b) Cohesive ground : cohesive ground with soft layer having small N-value has small bearing capacity which is determined by cohesion (C) and large displacement. However, cohesive ground with more than 20 of N-value is enough hard to utilize for bearing layer in general.

c) Rocky ground : large bearing capacity and less displacement

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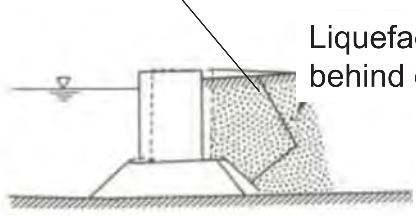
1. Foundation Planning in General

1.4 Example of Problems in Foundation

(i) Problems related to Liquefaction

Falling bridge

River Dike Collapse



Liquefaction behind quaywall

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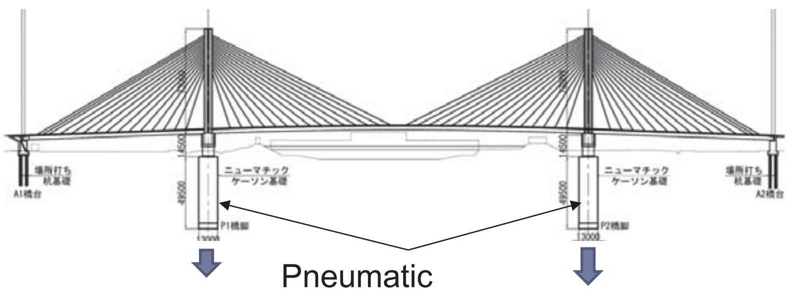
1. Foundation Planning in General

1.4 Example of Problems in Foundation

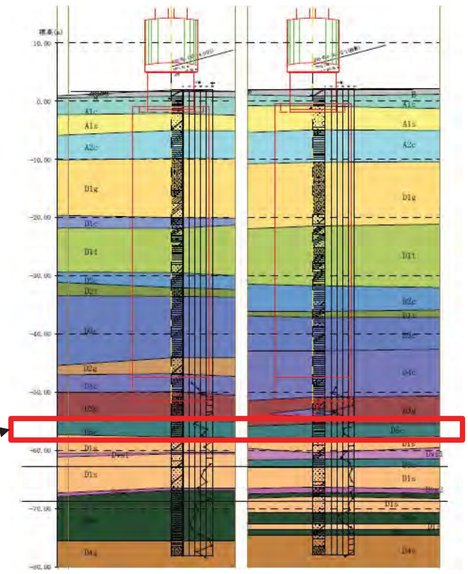
(ii) Problems related to Consolidation Settlement



4cm

Pneumatic Caisson

20cm



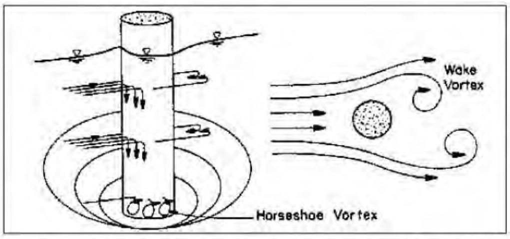
Main cause: consolidation settlement in cohesive soil layer under tip foundation

Countermeasure: Preloading etc.

1. Foundation Planning in General

1.4 Example of Problems in Foundation

(iii) Problems related to Local Scouring



Source: *Evaluating Scour at Bridges (2012 Fifth Edition), Hydraulic Engineering Circular No. 18 (HEC 18), FHWA, USA*

Schematic Representation of Scour at a Cylindrical Pier

1. Foundation Planning in General

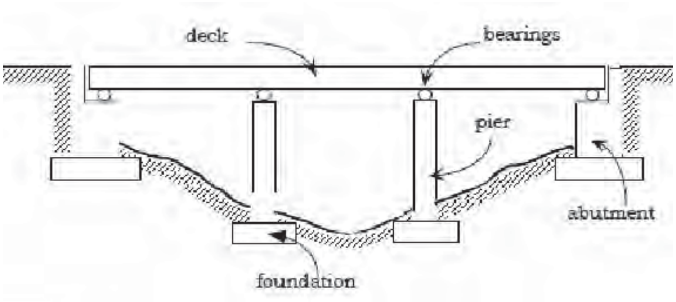
1.5 Foundation Type and its features

Foundation is classified into six main categories for foundation planning and design in Japan.

(i) Spread Foundation
(ii) Caisson Foundation
(iii) Pile Foundation
(iv) Steel Pipe Sheet Pile Foundation
(v) Diaphragm wall Foundation
(vi) Deep Foundation

(i) Spread Foundation

- The footing is directly built on shallow bearing ground without any special structures. The depth of bearing ground is normally at range from 5m to 10m.
- The size and shape of footing can be adjustable depending on the relationship between the loads from structure and bearing capacity of the ground.

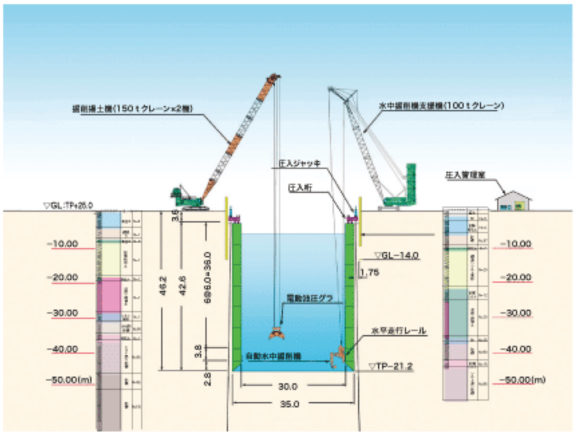


http://www.slideshare.net/soniafaisal/bridge-engin...

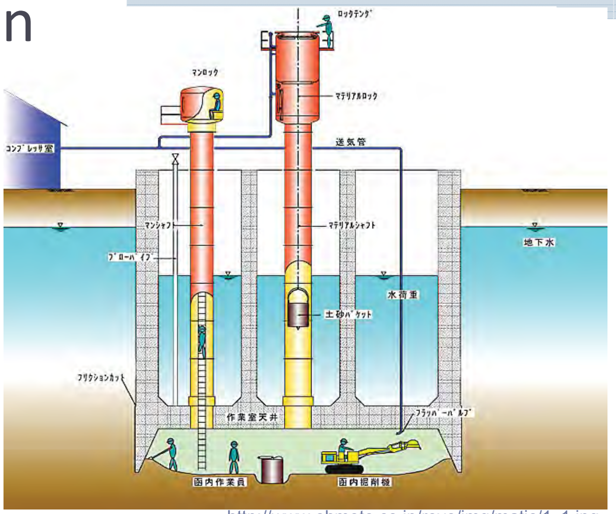
(ii) Caisson Foundation

- Caisson foundation is made with a well structure which inside is opened. The well is settled by draining out soils from the opening part to reach onto the bearing layer. It is classified to Open Caisson or Pneumatic Caisson.
- It is preferable if the depth of bearing layer is 10m – 30m, but applicable up to 60m depth.

(ii) Caisson Foundation



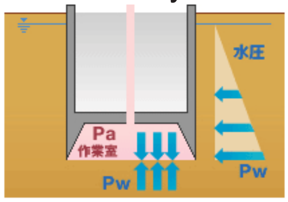
http://www.konoike.co.jp/et/detail/000151.html



http://www.ohmoto.co.jp/rovo/img/matic/1_1.jpg

Open Caisson :
The well fabricated on ground is installed and settled by excavating gradually through the opening by manually or machine.

Pneumatic Caisson :
The water pressure is restricted by the air pressure which is ventilated to the work room. High accuracy of setting work and easy to keep work schedule.
Cost is higher than open caisson because special facilities are required.



(iii) Pile Foundation

View of Pile Foundation (Cast-in-placed RC pile)

http://marutaidoboku.co.jp/works/05.html

http://www.ubaura.com/wp/?paged=16

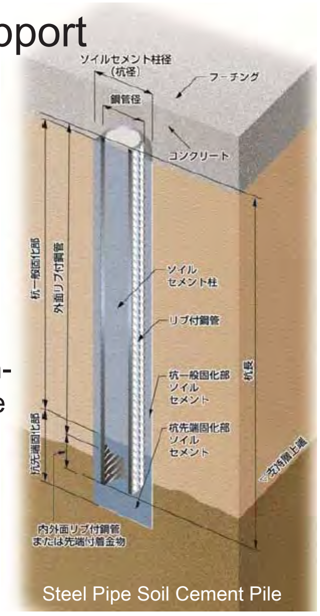
(iii) Pile Foundation

(1) Categorization of Pile Foundation

- Pile foundation is constructed by installing a slender structure to reach the bearing layer by driven or vibration or casting concrete for the hole excavated on the ground.
- Pile type is categorized by material type, support mechanism and construction method.

Categorization by Material

- Steel
  - Steel Pipe Pile
  - H Shaped Steel Pile
- Concrete
  - Precast Pile
    - RC Pile
    - PHC (Prestressed High-strength Concrete) Pile
  - Cast-in-Placed RC Pile
- Composite Type
  - Steel Pipe Soil Cement Pile
  - SC (Steel Composite Concrete) Pile



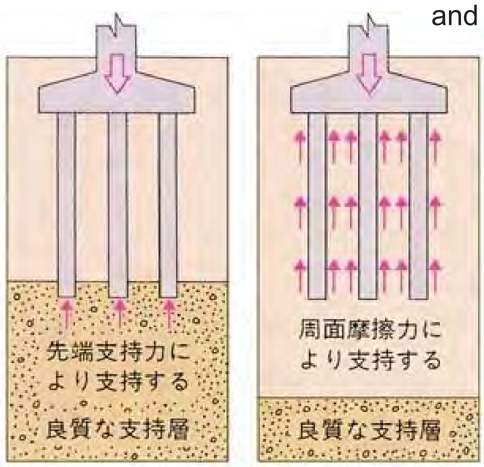
Steel Pipe Soil Cement Pile

(iii) Pile Foundation

Categorization by support mechanism

Pile Foundation

- Bearing Pile: Piles insert into firm bearing layer and vertical load will be mainly resisted by tip bearing.
- Friction Pile: Piles will not insert into firm bearing layer and vertical load will be mainly resisted by skin friction between piles and ground.



Bearing Pile   Friction Pile

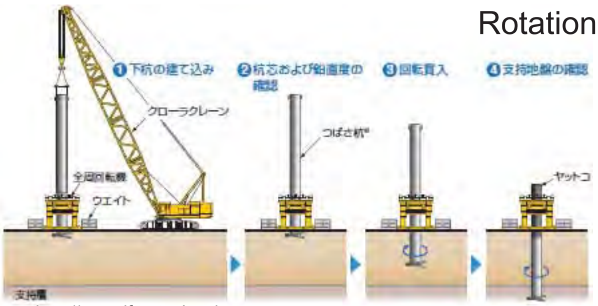
https://kotobank.jp/image/dictionary/nipponica/media/81306024015154.jpg

(iii) Pile Foundation

Categorization by construction method

- Precast Pile Method
  - Driven Pile
    - Driving Method
    - Vibro-Hammer Method
  - Embedding Pile
    - Excavate Inside Pile Method
    - Pre-boring Pile Method
    - Steel Pipe Soil Cement Method
  - Rotation Pile
- Cast-in-Placed Pile Method
  - Mechanical Boring
    - All casing Method
    - Reverse Circulation Drill Method
    - Earth Drill Method
  - Deep Foundation

Rotation Pile Method



http://www.jfe-steel.co.jp

(iii) Pile Foundation

(2) Cast-in-Placed RC Pile Method

- Cast in-placed RC pile is the piling method done by excavation with machine or manually and arranging re-bars and casting concrete in the hole.
- Cast-in placed RC pile can be classified into three method as All casing method, Reverse circulation drill method, Earth drill method.
- Three methods have similar features in terms of excavation and casting concrete under water. Some difference is in detail of excavating machine and stabilization of hole wall etc.

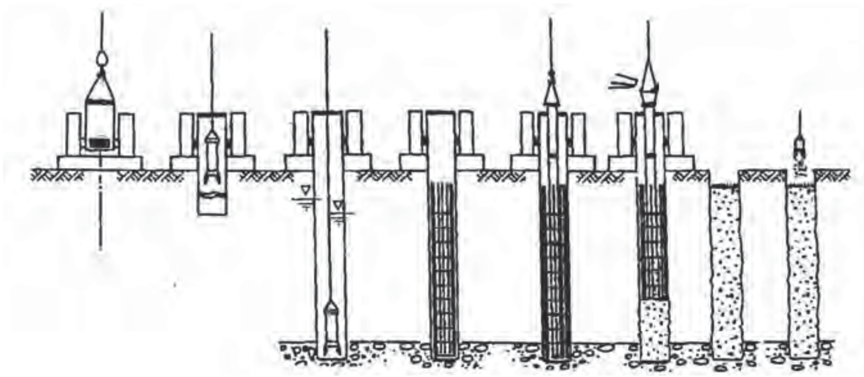
All Casing Method

i) Steel casing tube is installed by swinging, pressing for all length of pile.

ii) Soil in the hole is excavated out by hammer.

iii) Fabricated reinforcing cage is installed into the hole.

iv) Concrete is casting into the hole at same time of pulling out the casing tube.

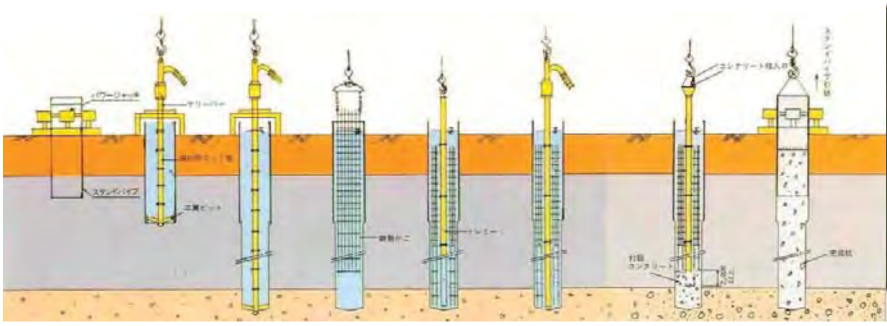


Reverse Circulation Drilling Method

i) Standpipe is installed. The hole is filled with water above ground water level in order to provide hydrostatic pressure from inside to protect failure of hole wall.

ii) Excavated soil and water is drained out by drilling pipe. The water is segregated with soil at the ground and reversed into the hole.

iii) Fabricated reinforcing cage is installed into the hole. Concrete is cast into the hole.



http://www.taiyo-kiso.co.jp/old/k13-14.htm

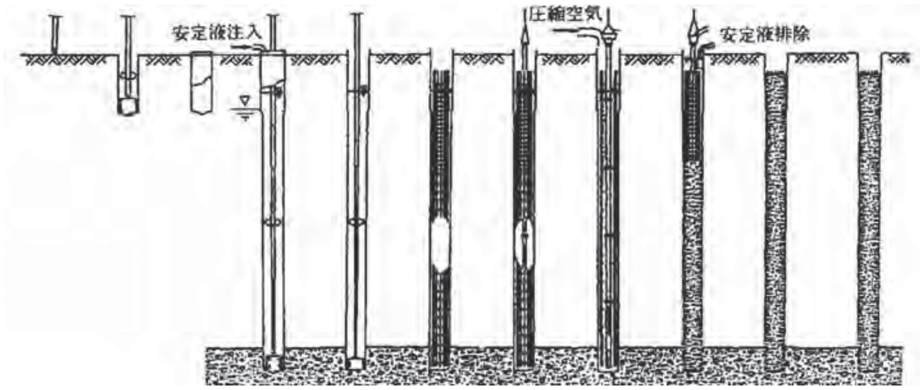
Construction Flow of Reverse Circulation Drill Method

Earth Drill Method

i) In order to prevent from failure at near surface, casing tubes are installed for top part. For lower depth, bentonite is used.

ii) The soil is excavated and removed by bucket.

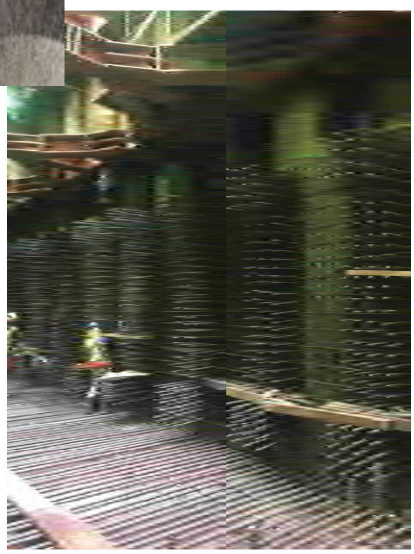
iii) Fabricated reinforcing bar is installed into the hole. Concrete is cast into the hole.



(iv) Steel Pipe Sheet Pile Foundation

View of Steel Pipe Sheet Pile Foundation (SPSP)

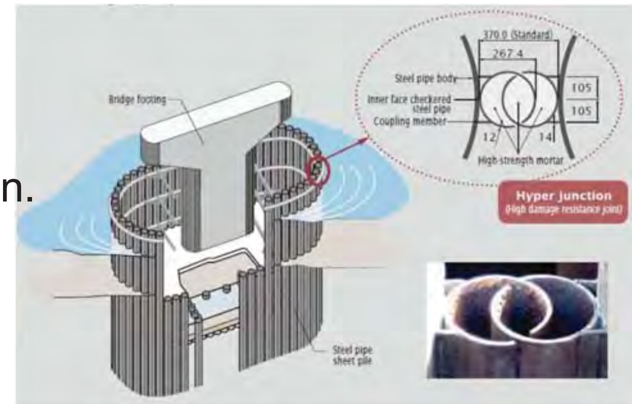
(Nippon Koei, New Thaketa Bridge Project)



(iv) Steel Pipe Sheet Pile Foundation

(1) General

- SPSP, in which steel pipe piles are provided with joints, are widely used for bridge for many long-span bridges and large structures.
- Filling the joint pipes of the steel pipe sheet piles with mortar and rigidly connecting to the footing will make a group of steel pipe sheet piles to behave as an integral foundation.
- Large rigidity and excellent work efficiency allow for rational design of foundation.

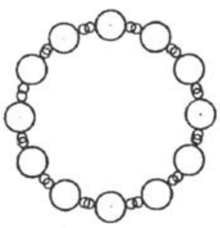


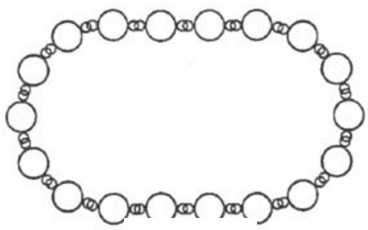
(JFE steel corporation major building material catalog)

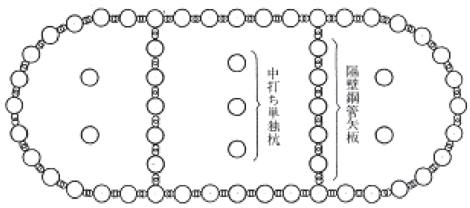
(iv) Steel Pipe Sheet Pile Foundation

(2) Type of Shape

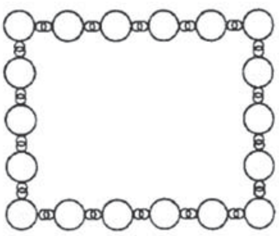
Several types of shape can be formed depending on pier column shape.

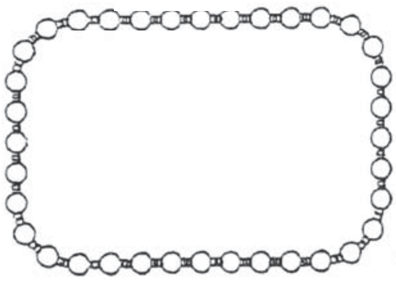

circle


oval



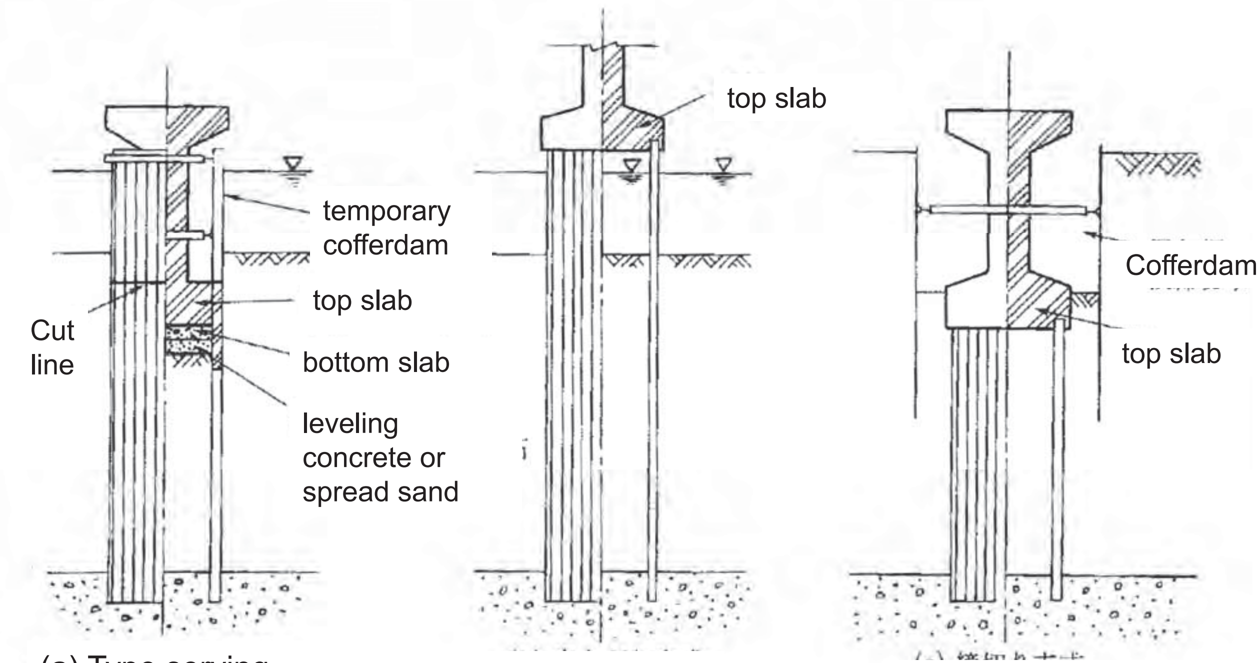
with diaphragm and supporting pile


square


Square with splay

(iv) Steel Pipe Sheet Pile Foundation

(3) Type of construction style



(a) Type serving temporary cofferdam

(b) Standing up Type

(c) Closing type

(iv) Steel Pipe Sheet Pile Foundation

(a)Type serving also temporary cofferdam :

SPSP is installed to be reach above water level for acting as cofferdam. The portion acting as cofferdam are cut out and removed after top slab and pier are constructed. It enable to shorten construction period minimize site occupation area.

(b) Standing up type :

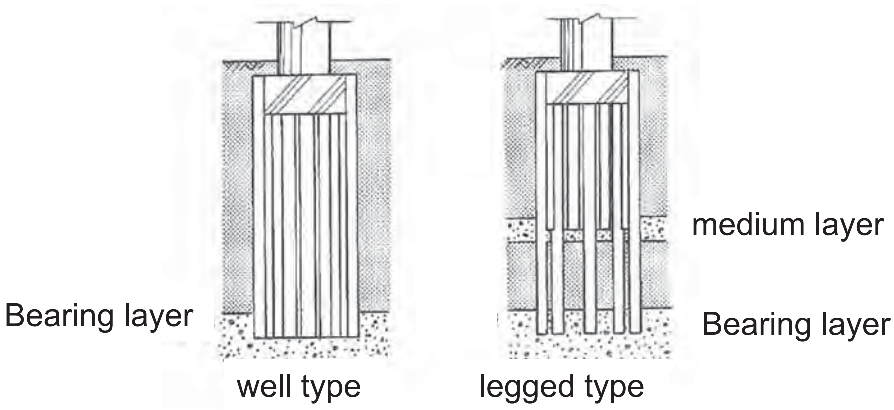
SPSP structure is used in permanent foundation only. Footing and pier will be constructed after installing the pile up to the water level. It is usually applied in river area or sea ports with unrestricted section of flow and clearance for ships crossing.

(c) Closing type : SPSP is constructed inside cofferdam arranged by sheet pile. It is seldom applied in recent.

(iv) Steel Pipe Sheet Pile Foundation

(4) Type of support

- **Well type** : All steel pipe sheet pile of the foundation is reached till bearing layer. It is common type of steel pipe sheet pile.
- **Legged type** : Half of steel pipe sheet pile is reached till bearing layer. Other pipe sheets is stopped in the medium layer with moderate bearing capacity.



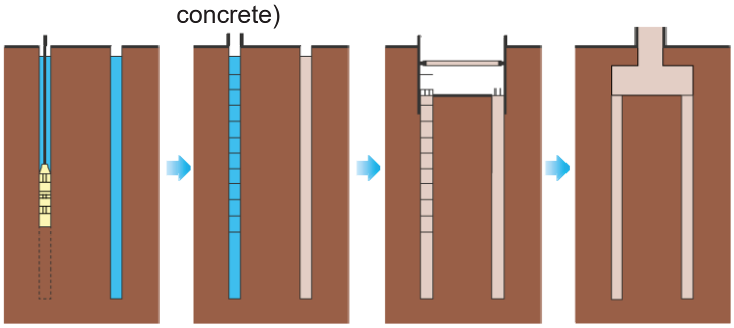
(v) Diaphragm Wall Foundation

View of Diaphragm wall foundation

http://www7b.biglobe.ne.jp/~renpeki/docs/tityu/b3_kiso.html

Construction Step

1. Excavation
2. Wall Construction (rebar cage, cast concrete)
3. Excavation (cofferdam, head treatment)
4. Footing Construction



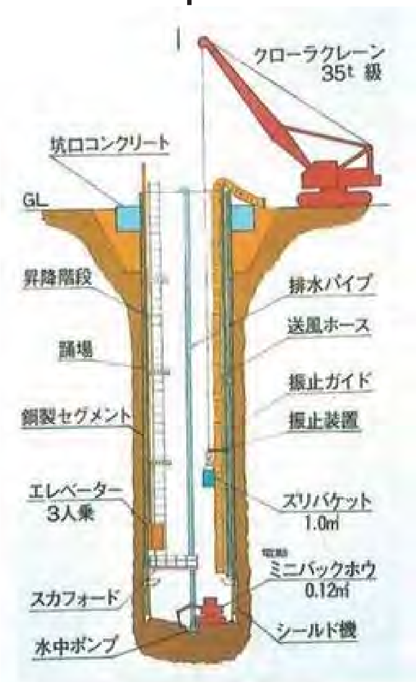
https://www.rncc.co.jp/tech/tc_6/30

(v) Diaphragm Wall Foundation

- Diaphragm wall method is the method to construct continuous underground wall by i) excavating ground soil with protecting failure of the wall by filling slurry in trench for ground stabilization. ii) Installing fabricated reinforcing bar and iii) casting concrete into the trench. The method is utilized as diaphragm wall foundation by constructing top slab on the wall top.
- Diaphragm wall is advantage for adherence with the ground. It induces large bearing capacity.
- Diaphragm wall foundation becomes large stiffness structure by applying closure section.

(vi) Deep Foundation

View of deep Foundation

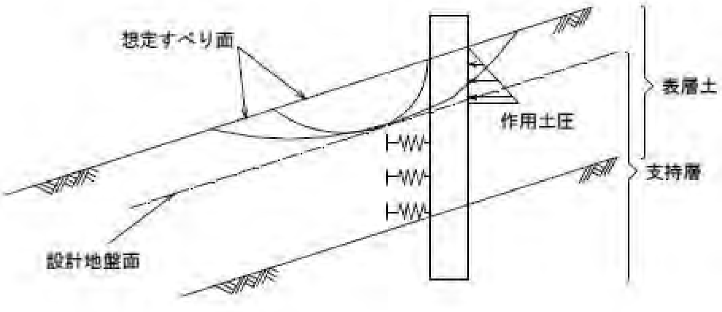


http://www.jasdim.or.jp/gijutsu/jisuberi_joho/sekk...

図4　主鉄筋の組み立て 3)

(vi) Deep Foundation

- Deep foundation is a kind of cast in-placed pile but by manual excavation.
- The excavation is done under drying condition so that excavated surface and condition of casted concrete could be visually confirmed. It also advantage for environment because of no-vibration and no-noise.
- Deep foundation is also utilized for the bridge locating at inclined land in mountainous area or at narrow site.
- It is not applicable for heavy inflow water or grounds that are likely to collapse



1. Foundation Planning in General

1.6 Referential Criteria of Applicability of Foundation Type

- In Japanese Specifications for Highway Bridges (JSHB) published by Japan Road Association, “Table for foundation selection” is attached as an appendix.
- The table is practically utilized for the foundation planning in Japan to select the suitable foundation type.

Applicability O: high △: moderate X: low

applied condition			spread foundation	Pile: driving – PHC piles・SC piles	Pile: driving – steel pipe piles – percussion method	Pile: driving – steel pipe piles – vibratory hammer method	Pile: boring – PHC piles・SC piles – final driving method	Pile: boring – PHC piles・SC piles – jetting and mixing method	Pile: boring – PHC piles・SC piles – concrete placement method	Pile: boring – steel pipe piles – final driving method	Pile: boring – steel pipe piles – jetting and mixing method	Pile: boring – steel pipe piles – concrete placement method	Pile: steel pipe pile soil cement piles	Pile: preboring method	Pile: cast-in-place piles method – all casing method	Pile: cast-in-place piles method – reverse circulation drill method	Pile: cast-in-place piles method – earth drill method	Pile: spinning method	Deep foundation: set pile deep foundation	Deep foundation: columnar deep foundation	caisson foundation: pneumatic	caisson foundation: open	steel pile sheet pile foundation	diaphragm wall foundation
ground condition	condition until bearing layer	There is a very soft layer in the middle layer or the vicinity of the surface layer	/	○	○	○	○	○	○	○	○	○	○	○	×	○	○	○	×	×	○	△	○	○
ground condition	condition until bearing layer	There is a very hard layer in the intermediate layer	/	△	△	△	○	○	○	○	○	○	○	○	△	○	×	○	○	○	○	△	△	○
ground condition	condition until bearing layer – There is gravel in the middle layer	gravel diameter less than or equal to 50mm	/	△	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
ground condition	condition until bearing layer – There is gravel in the middle layer	gravel diameter 50～100mm	/	△	△	△	△	△	△	△	△	△	○	○	△	×	○	○	○	○	○	○	△	△
ground condition	condition until bearing layer – There is gravel in the middle layer	gravel diameter 100～150mm	/	×	×	×	×	×	×	×	×	×	×	×	△	×	×	×	○	○	○	△	×	△
ground condition	condition until bearing layer	There is a ground to liquefaction	/	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	/	/	○	○	○	○
ground condition	bearing layer condition – depth	less than 5m	○	○	×	×	×	×	×	×	×	×	×	×	×	×	×	×	○	/	×	×	×	×
ground condition	bearing layer condition – depth	5～15m	△	○	○	○	○	○	○	○	○	○	○	○	○	△	○	○	○	○	○	○	△	△
ground condition	bearing layer condition – depth	15～25m	×	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
ground condition	bearing layer condition – depth	25～40m	×	○	○	○	○	○	○	○	○	○	○	○	○	○	△	○	△	△	○	○	○	○
ground condition	bearing layer condition – depth	40～60m	×	△	○	○	△	△	△	○	○	○	○	○	△	○	×	○	×	×	△	○	○	○
ground condition	bearing layer condition – depth	over 60m	×	×	△	△	×	×	×	×	×	×	△	△	×	△	×	○	×	×	×	△	△	△
ground condition	bearing layer condition – soil	sand ・ sand gravel (30≦N)	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○
ground condition	bearing layer condition – soil	cohesive soil (20≦N)	○	○	○	○	○	△	×	○	△	×	△	△	○	○	○	△	○	○	△	△	○	○
ground condition	bearing layer condition – soil	soft rock ・ hardpan	○	×	○	△	○	△	×	○	△	×	△	△	○	○	○	△	○	○	○	○	○	○
ground condition	bearing layer condition – soil	hard rock	○	×	×	×	×	×	×	×	×	×	×	×	△	△	△	×	○	○	△	×	×	△
ground condition	bearing layer condition	There is high possibility that the position of the support layer is not same depth, including; slope is large, irregularities of surface layer is heavy, etc.,	△	△	△	△	△	△	△	△	△	△	△	△	○	○	○	○	○	○	△	×	○	○
ground condition	groundwater condition	Groundwater level is near the ground surface	△	○	○	○	○	○	○	○	○	○	○	○	△	△	△	○	△	△	○	○	○	△
ground condition	groundwater condition	extremely large amount of sump water	△	○	○	○	○	○	○	○	○	○	△	△	△	△	△	○	×	×	○	○	○	△
ground condition	groundwater condition	there is artesian water that 2m deeper than the surface	×	○	○	○	×	×	×	×	×	×	×	×	×	×	×	○	×	×	△	△	○	×
ground condition	groundwater condition	groundwater flow rate over 3m/min	×	○	○	○	○	×	×	○	×	×	×	×	×	×	×	○	×	×	○	△	○	×
type of support		bearing pile	/	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	/	/	/	/	/
type of support		friction pile	/	○	○	○	×	×	×	×	×	×	○	×	○	○	○	×	/	/	/	/	/	/
construction condition	aquatic construction	water depth less than 5m	△	○	○	○	△	△	△	△	△	△	×	×	×	×	×	○	/	/	△	△	○	×
construction condition	aquatic construction	water depth over 5m	×	△	○	○	△	△	△	△	△	△	×	×	×	×	×	○	/	/	△	△	○	×
construction condition		narrowness of work space	○	△	△	△	△	△	△	△	△	△	△	△	△	△	△	△	○	○	△	△	×	△
construction condition		construction of batter pile	/	○	○	○	×	×	×	×	×	×	×	×	×	×	×	○	×	/	/	/	/	/
construction condition		effect of toxic gas	/	○	○	○	○	○	○	○	○	○	○	○	○	○	○	○	×	×	×	○	○	○
construction condition	surrounding environment	vibration and noise control	○	×	×	△	△	○	○	△	○	○	○	○	○	○	○	○	○	○	○	○	×	○
construction condition	surrounding environment	Effects on adjacent structures	○	×	△	△	△	○	○	△	○	○	○	○	○	○	○	○	△	△	△	△	△	○

2.Substructure Planning in General

2.1 Type of Bridge Pier

- Pier is the substructure which has roles to support superstructure between abutments and to transfer loads from superstructure to foundation ground.
- Pier type shall be decided with comprehensive aspects such as economic efficiency, landscape, construction and maintenance workability.
- Type of bridge pier is classified:

✓Pile bent type

✓Reverse-T type - Wall type

- Beam type

✓Rigid frame type

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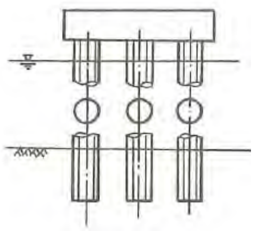
2.1 Type of Bridge Pier

Type	Condition to be applied	Feature
Pile bent pier   http://www.ktr.mlit.go.jp/kawakoku/genba/7wangan_ariake/2007_12_01.htm	River bridge which cofferdam work is not suitable.	・can reduce construction time and cost because of unnecessity of cofferdam work. ・have concern about obstruction of water flow by flown debris such as trees. ・Due to less stiffness of piers against seismic force, it requires enough width for bridge seat.

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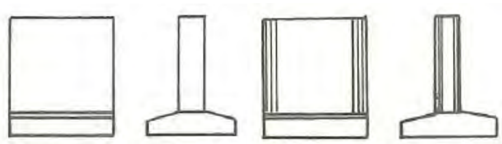
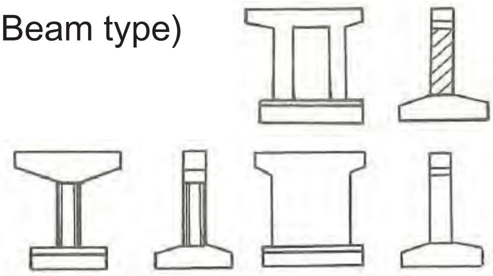
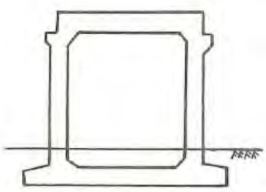
2.1 Type of Bridge Pier

Type	Condition to be applied	Feature
Reverse-T (Wall type)   http://www.ubaura.com/history2015.html	River bridge, flyover, etc.	・Not provide beam part so that it eases construction. ・Concrete volume is lager than Reverse-T beam type, thus heavy structure. ・For river bridge, column shape of circular or oval type is normally applied.

2.1 Type of Bridge Pier

Type	Condition to be applied	Feature
Reverse-T (Beam type)   http://www.sato-kogyo.co.jp/jusyo02.html	River bridge, flyover, etc.	・Concrete volume is smaller than Reverse-T wall type. ・The shape of column is slenderness compare with wall type. ・The space under beam can be utilized. ・For river bridge, column shape of circular or oval type is normally applied.

2.1 Type of Bridge Pier

Type	Condition to be applied	Feature
Rigid frame   http://blog-imgs-62.fc2.com/k/p/f/kpfrs/00000601.jpg	Flyover, viaduct. railway viaduct	・Because of stiffness efficiency, it enable to reduce structural dimension. ・The space under girder is utilized specially at urban area, example parking lots, supermarket, restaurant etc.

2.1 Type of Bridge Pier

Various type of pier

http://portal.nifty.com/kiji/130213159554_1.htm

2.1 Type of Bridge Pier

Various type of pier

http://portal.nifty.com/kiji/130213159554_1.htm

2.Substructure Planning in General

2.2 Type of Bridge Abutment

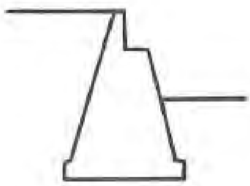
・Abutment is the substructure which support super-structure at bridge in ends. Also, it support earth pressure worked on wall from the back side.

・Abutment type need to be decided with comprehensive aspects such as economic efficiency, landscape, construction and maintenance workability.

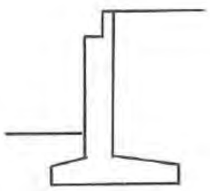
・Abutment type is classified such as;

- Gravity type
- Reverse-T type
- Counterfort type
- Rigid frame type

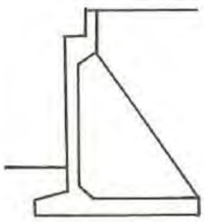
2.2 Type of Bridge Abutment

Type	Preferable Height	Feature
Gravity wall   http://www.shortspansteelbridges.org/steel-solutions/substructures.aspx	H≦ 3～6m	・The structure is formed by concrete without reinforcement. Only compressive stress is worked on the structure. ・Because of simple structure, construction work is easy. ・Due to large structural volume, good support ground is required and land occupation and excavation of soil become large.

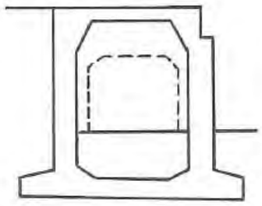
2.2 Type of Bridge Abutment

Type	Preferable Height	Feature
Reverse- T type   http://www.furano.ne.jp/furanodouro/bbs/diary.cgi	5m≦ H≦15m	・The structure is formed with reinforced concrete. ・The type has economic efficiency if the height exceeds 5m. ・Self weight is small and stability can be obtained by weight of backfilling soil.

2.2 Type of Bridge Abutment

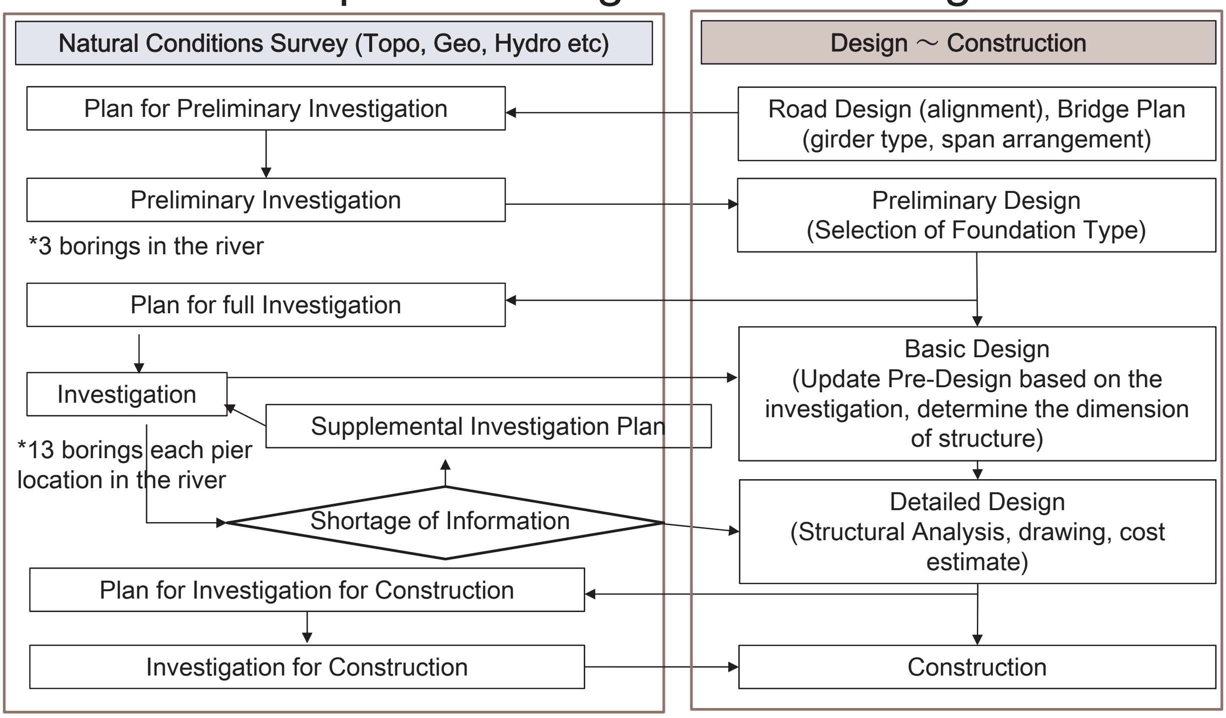
Type	Preferable Height	Feature
Counterfort type   http://www.afrostructures.co.za/currentprojects.asp	12m≦ H≦15m	・The type has economic efficiency if the height exceeds 10m. ・It needs to take account for concrete casting work and rebar arrangement work at counterfort wall. ・Also attention requires for compacting backfilling work around counterfort walls.

2.2 Type of Bridge Abutment

Type	Preferable Height	Feature
Rigid frame   http://go-isesaki.goisesaki.3zoku.com/roads_354_tone_bridge.htm	10m≦ H≦15m	・The type can be suitable if road needs to cross at abutment location.

3.Plan of Substructure and Foundation for Bago River Bridge

3.1 Overall Step for Investigation and Design



3.2 Topographical condition

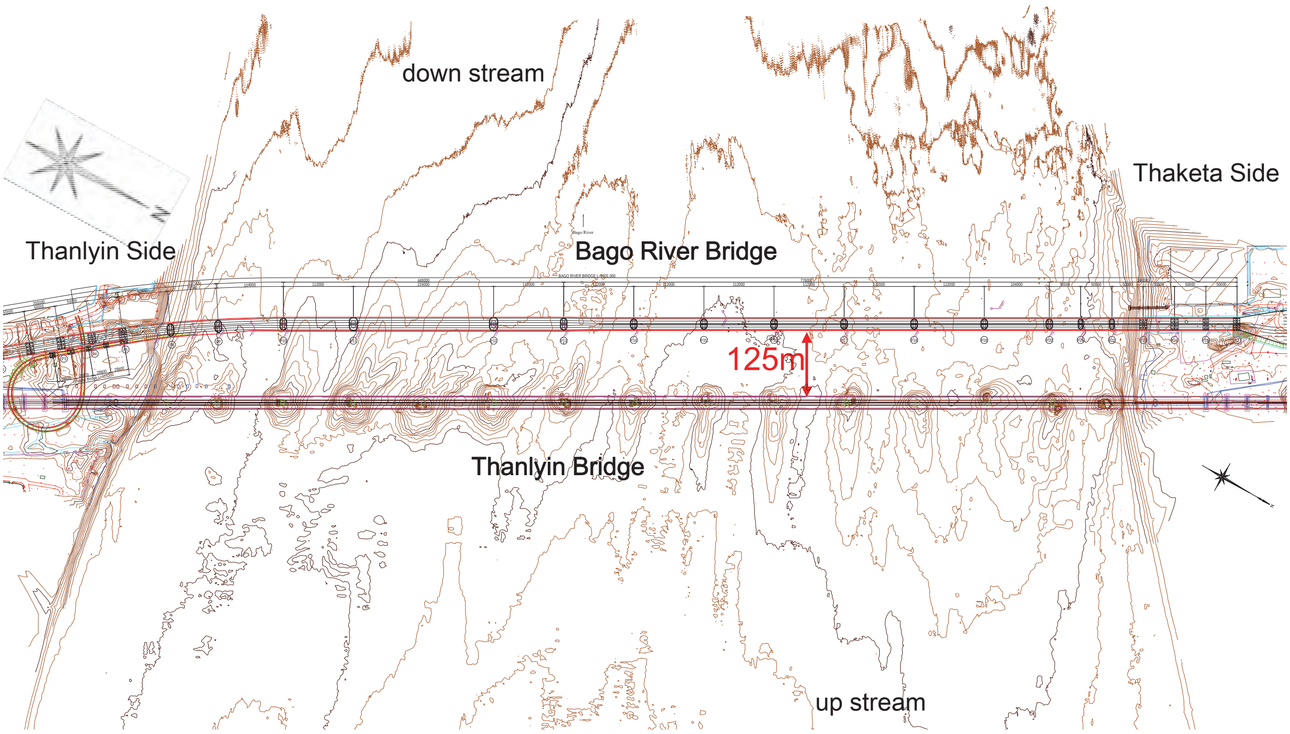
(1) Topography

As for the project area, the proposed Bago River Bridge is located at the flood plain deposit area of Bago River, thus the area is dominated by flat lying topography in general.

Bago River shows the old age stage of meandering. The river process of deposition is dominant than erosional.

3.2 Topographical condition

2) Plan with Horizontal Alignment of Bago River Bridge in River Section

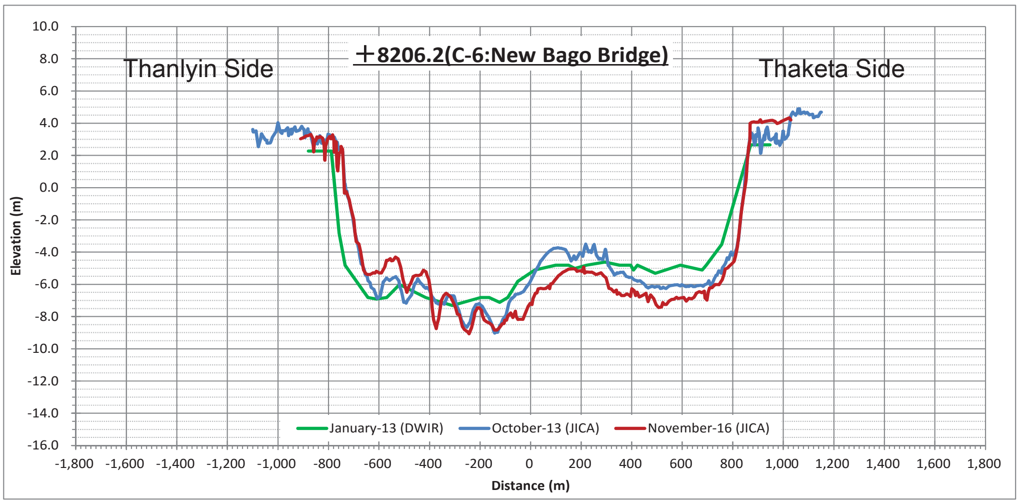


3.2 Topographical condition

3) River Bed Profile

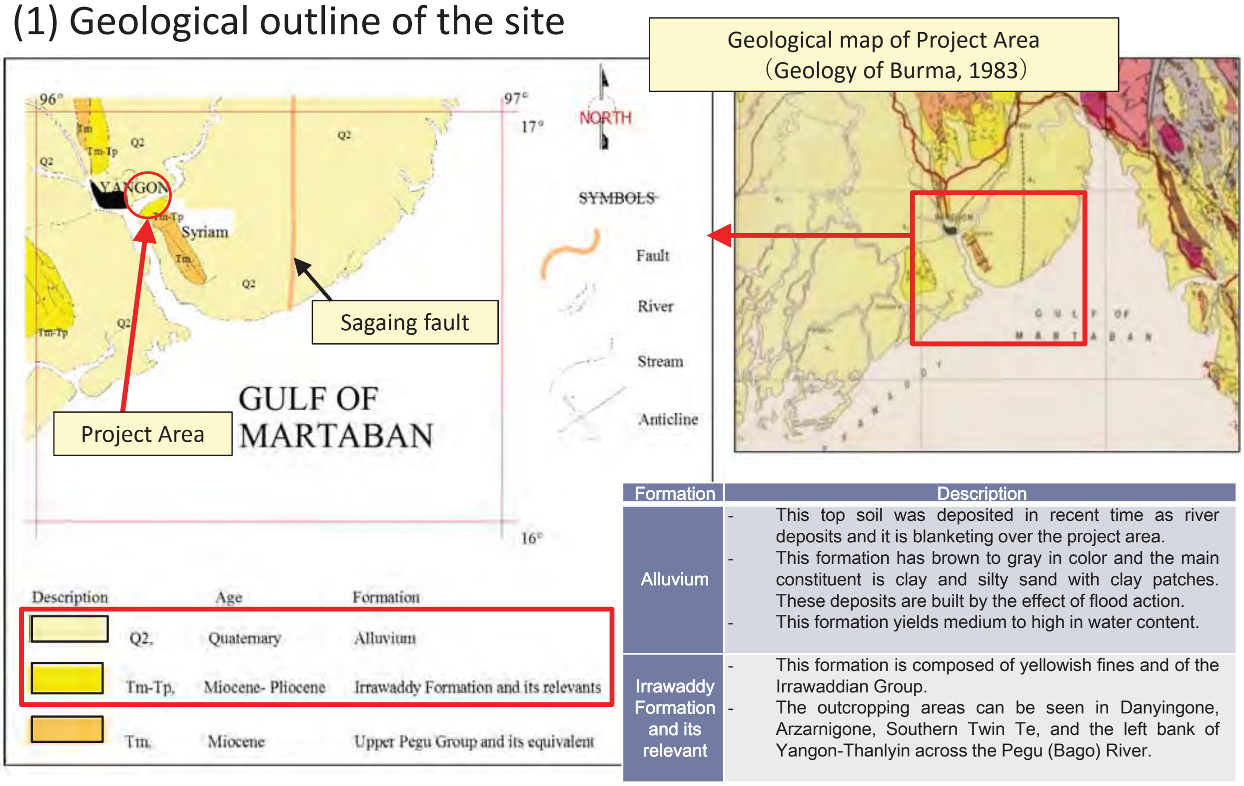
The fluctuations of the cross-sectional shape of the river at the location of the new bridge in recent years are shown in the figure.

- Its shapes indicate the trend toward increasing erosion at the right bank, while increasing deposition at the left bank.
- There are no obvious differences of riverbank lines at both banks in 2013 and 2016.



3.3 Geotechnical condition

(1) Geological outline of the site



Formation	Description
Alluvium	- This top soil was deposited in recent time as river deposits and it is blanketing over the project area. - This formation has brown to gray in color and the main constituent is clay and silty sand with clay patches. These deposits are built by the effect of flood action. - This formation yields medium to high in water content.
Irrawaddy Formation and its relevant	- This formation is composed of yellowish fines and of the Irrawaddian Group. - The outcropping areas can be seen in Danyingone, Arzarnigone, Southern Twin Te, and the left bank of Yangon-Thanlyin across the Pegu (Bago) River.

3.3 Geotechnical condition

(2) Ground Conditions and Bearing Layer

- By boring results of soil investigation, the project area is made up of alluvial deposit of clay, silty sand and clayey sand.

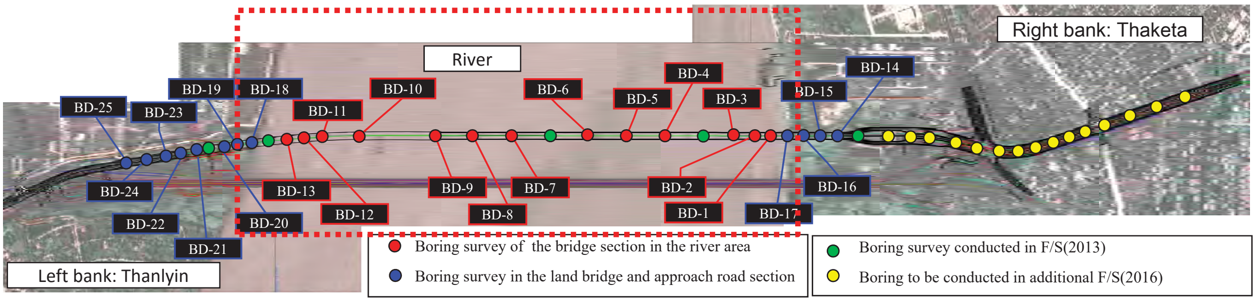
- Bearing Layer

✓Sand Layer: N value of 30 or more (Clayey SAND-II)

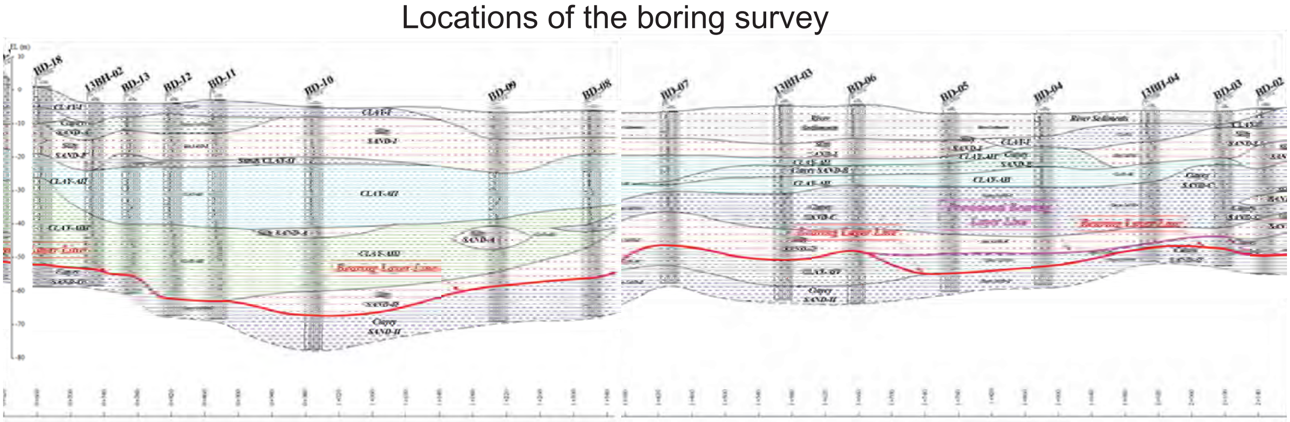
✓Cohesive Soil Layer: N value of 20 or more (CLAY-AIV, CLAY-III, CLAY-IV)

✓In addition, Clayey SAND-I distributed from the right bank of the river bed to Taketa area can be evaluated as a provisional bearing layer, since N values of 30 or more were continuously confirmed.

(2) Ground Conditions and Bearing Layer



Locations of the boring survey



Soil Profile with Bearing Layer

3.3 Geotechnical condition

(3) Geotechnical Design Parameters for River Section

No.	Soil Name	Representative N Value	Unit Weight			Internal Friction Angle	Cohesive Strength	Deformation Modulus
			γt (kN/m³)	γsat (kN/m³)	γ' (kN/m³)	φ (°)	c (kN/m²)	E50 (kN/m²)
1	Silty SAND-River Sediments	3	17.0	18.0	8.0	29	-	1200
2	CLAY-I	1	17.5	17.5	7.5	-	10	900
3	Clayey SAND-A	3	17.5	18.5	8.5	28	-	1200
4	Silty SAND-I	13	17.0	18.0	8.0	33	-	5200
5	Sandy CLAY-II	9	17.5 Same values as CLAY-AII	17.5	7.5	-	54	6300
6	CLAY-AII	7	17.5	17.5	7.5	-	42	4900
7	Clayey SAND-B	13	17.0	18.0	8.0	32	-	9100
8	Silty SAND-A	25	17.0	18.0	8.0	33	-	17500
9	CLAY-AIII	18	18.0	18.0	8.0	-	108	12600
10	Clayey SAND-C	20	17.0	18.0	8.0	33	-	14000
11	Silty SAND-II	30	17.0	18.0	8.0	34	-	21000
12	Clayey SAND-I	35	19.0	20.0	10.0	34	-	24500
13	CLAY-AIV	30	18.0	18.0	8.0	-	180	21000
14	Clayey SAND-II	50	19.0	20.0	10.0	35	-	35000

3.3 Geotechnical condition

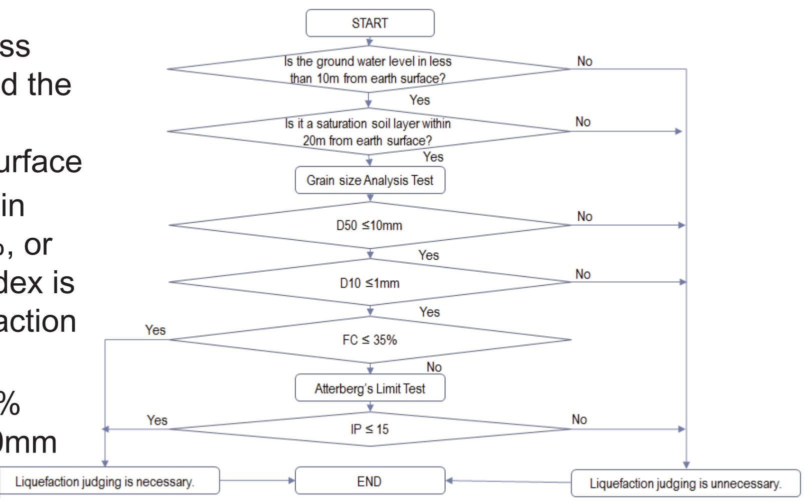
(4) analysis of liquefaction possibility

Initial Assessment of Potential of Liquefaction

1) The groundwater level is in less than 10m from earth surface, and the saturation soil layer exists in the depth within 20m from ground surface

2) The soil layer whose fine-grain fraction content is less than 35%, or the soil layer whose plasticity index is less than 15 even if fine-grain fraction content exceeds 35%

3) The soil layer which D50 [50% particle diameter] is less than 10mm and D10 [10% particle diameter] is less than 1 mm.



Flow Chart of Initial Assessment of Potential of Liquefaction

If all yes, need to calculate FL (resistivity to liquefaction) for judgement.
If FL is less than 1.0, its soil layer will be judged to occur the liquefaction.

3.3 Geotechnical condition

(4) analysis of liquefaction possibility

Calculation of FL in accordance with JSHB

$F_L = R/L$ (formula) - (1)

$R = c_w R_L$ (formula) - (2)

$L = r_d k_{hgL} \sigma_v / \sigma_v'$ (formula) - (3)

$r_d = 1.0 - 0.015x$ (formula) - (4)

$k_{hgL} = c_z k_{hgL0}$ (formula) - (5)

$c_w = 1.0$ (formula) - (6)

Where,

F_L = Resistivity to liquefaction
R = Dynamic shear strength ratio
L = The earthquake shear stress ratio
c_w = Correction factor by earthquake vibration properties. In here, C_w=1.0 for level-1 earthquake.
R_L = The repetition triaxiality strength ratio
r_d = Reduction coefficient of the depth direction of the earthquake shear stress ratio
k_{hgL} = Design horizontal seismic intensity of the ground surface to use for a judgment of the liquefaction
c_z = Seismic zone factor. Here, it was set with 1.0.
k_{hgL0}= Standard value of the design horizontal seismic intensity of the ground surface to use for a judgment of the liquefaction. In here, it is 0.18 in soft ground and for level-1 earthquake.
σv = The total pressure exerted by earth
σv' = The effective overburden pressure
x = Depth from an earth surface

The repetition triaxiality strength ratio "RL" is computed by the formula (7).

$R_L = 0.0882\sqrt{N_a/1.7}$ (Na<14)
$R_L = 0.0882\sqrt{N_a/1.7} + 1.6 \times 10^{-6} \cdot (N_a - 14)^{4.5}$ (Na≧14)
Formula - (7)

<In the case of a sandy soil>

$N_a = c_1 N_1 + c_2$ formula - (8)

$N_1 = 170N/(\sigma'_{vb} + 70)$ formula - (9)

$c_1 = 1$ (0%≦FC<10%)
$c_1 = (FC + 40)/50$ (10%≦FC<60%)
$c_1 = FC/20 - 1$ (60%≦FC)
Formula - (10)

$c_2 = 0$ (0%≦FC<10%)
$c_2 = (FC - 10)/18$ (10%≦FC)
Formula - (11)

Where,
R_L ＝The dynamic shear strength ratio N＝N-value
N1 ＝N-value which considerably converted into the effective overburden pressure 100kN/m2
σ'$_{vb}$ ＝The effective overburden pressure in the depth from the earth surface at the time of the examination (kN/m2)
c1,c2 ＝The correction factor of N-value by the content for an infinitesimal grain
FC ＝The content for an infinitesimal grain（%） D50(mm) ＝50% particle size

3.3 Geotechnical condition

(4) analysis of liquefaction possibility

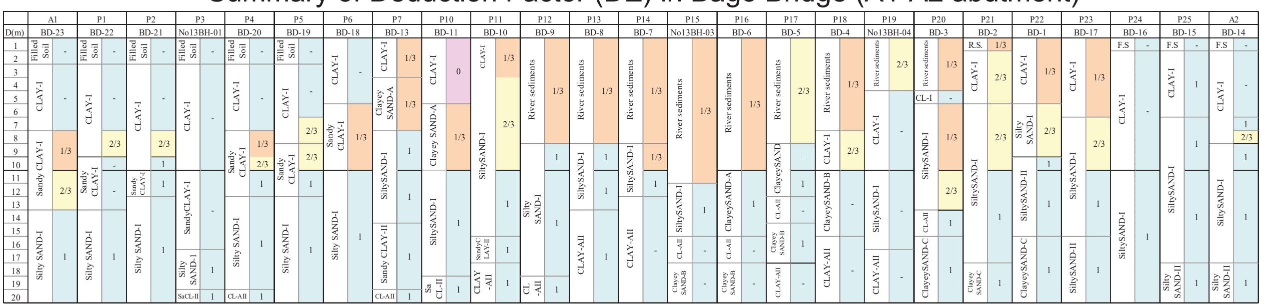
How to consider liquefaction to the design?

Deduction Factor (DE) on Soil Modulus

FL	Depth from an earth surface x(m)	R (The dynamic shear strength ratio) R≦0.3	0.3<R
F_L≦1/3	0≦x≦10	0	1/6
	10<x≦20	1/3	1/3
1/3<F_L≦2/3	0≦x≦10	1/3	2/3
	10<x≦20	2/3	2/3
2/3<F_L≦1	0≦x≦10	2/3	1
	10<x≦20	1	1

DE x Coefficient of Subgrade Reaction
DE x Maximum shaft resistance of soil layer

Summary of Deduction Factor (DE) in Bago Bridge (A1-A2 abutment)



3.4 Hydrological Survey and Hydraulic Analysis

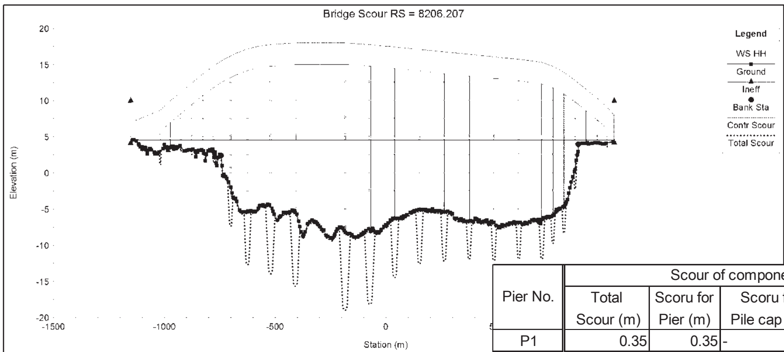
(1) Design Discharge and Design Water Levels

From the hydraulic analyses, the design high water level and discharge are determined as shown in the table.
Regarding the discharge, most of the total discharge is decided by the component of tidal flow other than the river's own flow (upland flow) from the catchment area, for too large of a tidal variation.

Item	Design Conditions
Design discharge	16,169 m^3/s (100-year return period)
Design high water level (HWL)	

Load combination	Supposition	Water level (MSL＋m)	River flow (m/s)
Normal	Full/low tide of spring tide	+3.18／-2.39	0
Wind	Highest HWL	+4.99	0
Collision at navigation span	Full tide of spring tide	+3.18	0
Collision at side span	Maximum river flow at flood of 100year return period	+2.53	1.19
Earthquake	Normal water level	+0.29	0.60
During construction	5year return period	+4.34	0.65

(2) Scour Estimation



Scouring Computation Result at New Bago Bridge by the CSU Equation

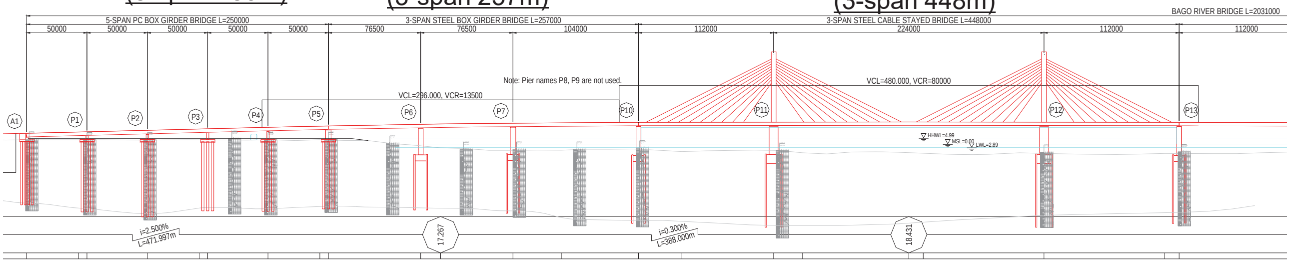
Pier No.	Scour of components				Rivebed Elevation (MSL+m)	Water Depth (m)	Mean Velocity (m/s)	Pile top Elevation (MSL+m)	Scoured Level (MSL+m)
	Total Scour (m)	Scoru for Pier (m)	Scoru for Pile cap (m)	Contraction Scour (m)					
P1	0.35	0.35	-	0.00	4.30	0.29	0.02	3.55	3.95
P2	0.36	0.36	-	0.00	4.30	0.29	0.02	3.49	3.94
P3	0.37	0.37	-	0.00	4.30	0.29	0.02	3.44	3.93
P4	0.20	0.20	-	0.00	4.30	0.29	0.02	3.49	4.10
P5	0.32	0.32	-	0.00	4.30	0.29	0.02	3.51	3.98
P6	3.86	3.15	0.36	0.35	-1.72	6.31	0.78	-3.45	-5.58
P7	2.34	1.01	0.99	0.35	-5.35	9.94	0.78	-3.45	-7.69
P10	6.72	5.80	0.58	0.35	-4.55	9.14	0.88	-9.20	-11.27
P11	6.72	5.53	0.84	0.35	-5.41	10.00	1.00	-9.20	-12.13
P12	5.71	4.25	1.11	0.35	-7.96	12.55	1.06	-9.20	-13.67
P13	5.46	4.14	0.97	0.35	-8.02	12.61	1.01	-9.20	-13.48
P14	5.14	4.03	0.76	0.35	-6.28	10.87	1.01	-8.06	-11.42
P15	5.74	4.73	0.66	0.35	-5.09	9.68	0.89	-8.06	-10.83
P16	5.08	4.11	0.63	0.35	-5.26	9.85	0.92	-8.06	-10.35
P17	2.99	2.28	0.36	0.35	-6.70	11.29	0.92	-8.06	-9.69
P18	3.00	2.12	0.53	0.35	-6.99	11.58	0.98	-8.06	-9.99
P19	2.89	2.09	0.45	0.35	-6.88	11.47	0.97	-8.06	-9.77
P20	2.97	2.00	0.62	0.35	-6.55	11.14	0.97	-7.28	-9.52
P21	2.40	1.71	0.34	0.35	-6.15	10.74	0.79	-7.55	-8.55
P22	2.86	2.51	-	0.35	-4.61	9.20	0.79	-7.59	-7.47
P23	2.01	1.66	-	0.35	-0.05	4.64	0.79	-2.39	-2.06
P24	0.13	0.13	-	0.00	4.11	0.48	0.01	3.73	3.98
P25	0.13	0.13	-	0.00	4.04	0.55	0.01	3.78	3.92

3.5 Confirmation of overall bridge plan

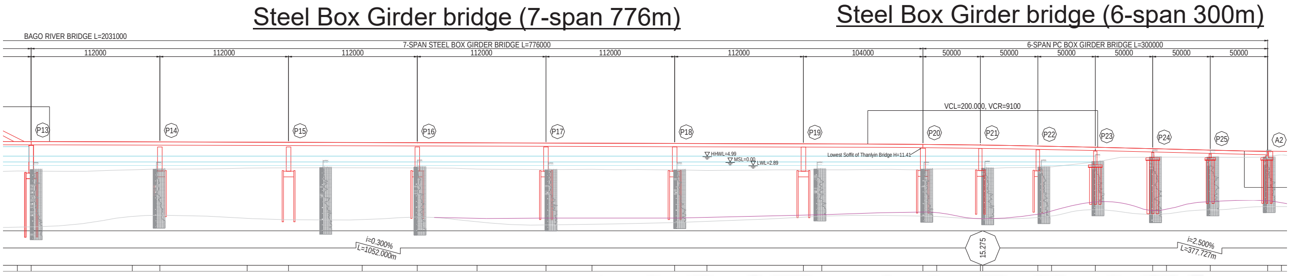
- Bridge foundation shall be planned by taking into account of overall bridge plan.
- In consideration of the hydrological advantage and safety for the vessel, the pier arrangement of Bago River Bridge was allocated on the line-of-sight of the existing Thanlyin Bridge. Although Bago River is relatively shallow, middle-class vessel runs through the abyss near the Thanlyin side.
- Navigation height is determined by the lowest soffit of Thanlyin Bridge at the P20 pier location of Bago River Bridge where the vertical alignment is lowest at navigation channel.

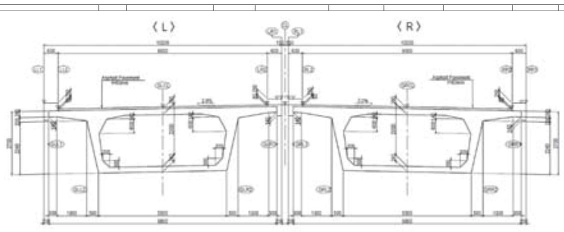
3.5 Confirmation of overall bridge plan

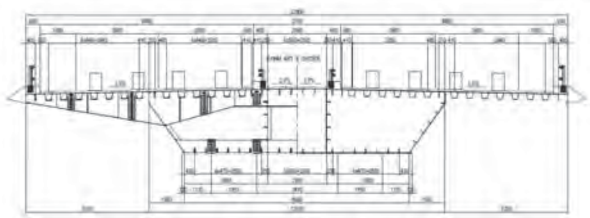
PC Box Girder bridge (5-span 250m) | Steel Box Girder bridge (3-span 257m) | Steel Cable Stayed bridge (3-span 448m)



Steel Box Girder bridge (7-span 776m) | Steel Box Girder bridge (6-span 300m)







3.6 Selection of foundation and Substructure

1.Preliminary Design Stage

- ☐ Foundation Type in the River Section
- ☐ Foundation Type on the Land Section
- ☐ Shape of Pier and Overhang
- ☐ Type of Abutment   with dimension

(1) Foundation Type in the River Section

Items to be considered for selection of foundation type
in the river section

- ✓ Maximum water depth at proposed bridge site is deeper than 10m.
- ✓ Local scouring
- ✓ The foundation must be able to support a large vertical load.
- ✓ Supporting layer will exist at deep location around EL-60m.

3.6 Selection of foundation and Substructure

1. Preliminary Design Stage (1) Foundation Type in the River Section

Applicable Condition \ Foundation Type			Cast- in-place Concrete Pile	PHC / SC Pile	Steel Pipe Pile	Diaphragm wall	Steel Pipe Sheet Pile	Caisson
Condition of Construction	Temporary Jetty	Depth < 5 m	△	○	○	×	○	△
		Depth > 5 m	△	△	○	×	○	△
	Environment	Vibration Noise	○	×	×	○	△	○
		Impact on Adjacent Structure	○	×	△	○	△	△
	Loading	Normal	○	○	○	○	○	○
		Large	○	×	○	○	○	○
Ground Condition	Depth of Supporting Layer	< 5 m	△	×	×	×	×	×
		5~15 m	○	○	○	△	△	○
		15~25 m	○	○	○	○	○	○
		25~40 m	○	○	○	○	○	○
		40~60 m	○	△	○	○	○	○
		>= 60 m	△	×	△	△	△	△
	Soil Condition	Clay (20 =< N)	○	○	○	○	○	○
		Sand/Gravel (30 =< N)	○	○	○	○	○	○

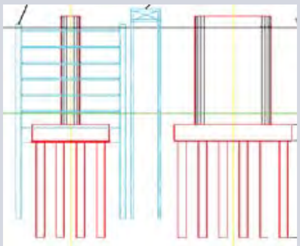
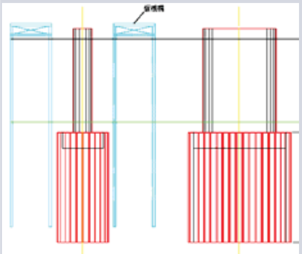
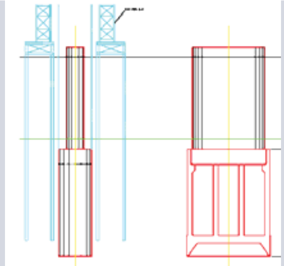
Other considerations:

- ✓ Pile Bent Type should be avoided because of weakness against scouring and horizontal seismic force.
- ✓ Steel Pipe Pile with cofferdam is obviously higher cost than CIP RC Pile with cofferdam, so it is omitted from alternatives.

i. CIP RC Pile with cofferdam
ii. SPSP
iii. Caisson

3.6 Selection of foundation and Substructure

1. Preliminary Design Stage (1) Foundation in the River Section

	Cast-in-place Concrete Pile	Steel pipe Sheet Pile Foundation	Concrete Caisson
Image			
Workability in Water	Inferior	Superior	Moderate
Work Period	Moderate	Superior	Moderate
Against Ship Collision	Superior	Superior	Superior
Against Scouring	Superior	Superior	Superior
Construction Safety	Moderate	Superior	Superior
Cost	Inferior	Moderate	Moderate
New Technology	Not New	New	Not New
Evaluation	Not Recommendable	Recommendable	Not Recommendable

3.6 Selection of foundation and Substructure

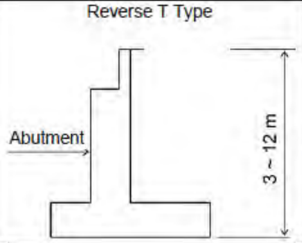
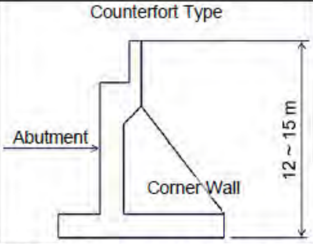
1.Preliminary Design Stage

(2) Foundation Type on the land section

Cast-in-Placed Pile Foundation by reverse circulation drilling method with casing pipe was selected.

- ✓ Easy constructability on ground and procurement of materials/equipment
- ✓ Widely used in Myanmar

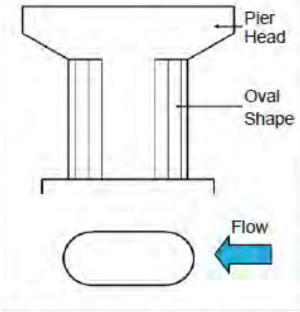
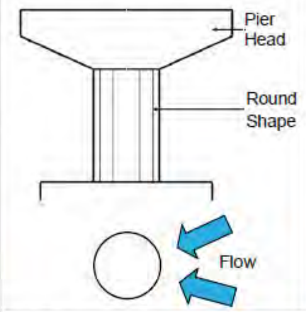
(3) Type of Abutment

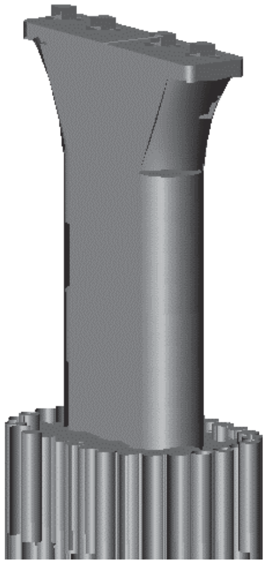
	Reverse T Type	Counterfort Type
Abutment Type		
Feature	It is very common and has an economical shape.	It is difficult to construct corner wall. It is not a common shape. Although box type is also considered, it is not an economical shape.

3.6 Selection of foundation and Substructure

1.Preliminary Design Stage

(4) Shape of Pier Column and Overhang

	Oval	Round
Substructure Type		
Feature	It is applied to the river bridge. Oval shaped pier is set parallel to the water flow in order to keep smooth water flow.	It is applied to the river bridge. When the direction of the river flow is not fixed such as in the river junction, it is applied.



3.6 Selection of foundation and Substructure

2.Basic Design Stage

- ☐ Footing Top Elevation and Pile Tip Elevation
- ☐ Foundation type at Riverfront Pier
- ☐ Diameter and thickness of SPSP
- ☐ Diameter of CIP Pile of Piers
- ☐ Diameter of CIP Pile of Abutment

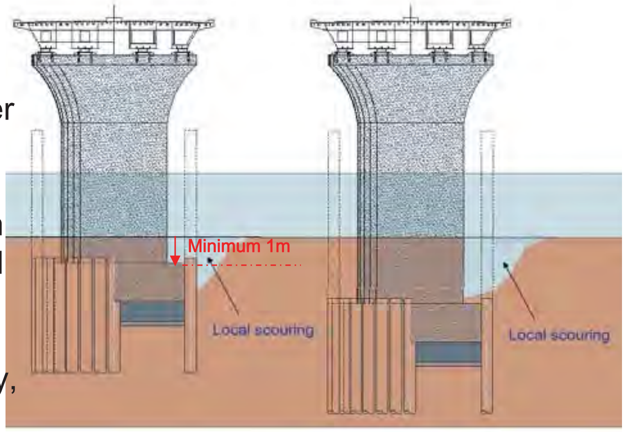
3.6 Selection of foundation and Substructure

2.Basic Design Stage

(1) Footing Top Elevation and Pile Tip Elevation

Footing Top Elevation

For the design of the SPSP, in general, deeper setting of footing below the riverbed may require a thicker steel pipe and/or higher grade pile due to larger displacement and stress during construction.
Footing top elevation is set to more than 1 m from the lowest elevation of existing riverbed among grouped piers.
Projection of the footing above the riverbed after local scouring will be allowed and finally, the stability during ordinary and earthquake conditions will be considered in the design.

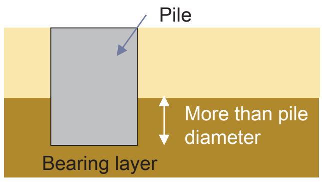


Allow injection after scouring

Embed below ground after scouring

Pile Tip Elevation

Reliable support soil layer N-value greater than 30 for sand soil and 20 for clay soil.
Pile tip is set into the bearing layer to more than the length of the diameter of pile.



3.6 Selection of foundation and Substructure

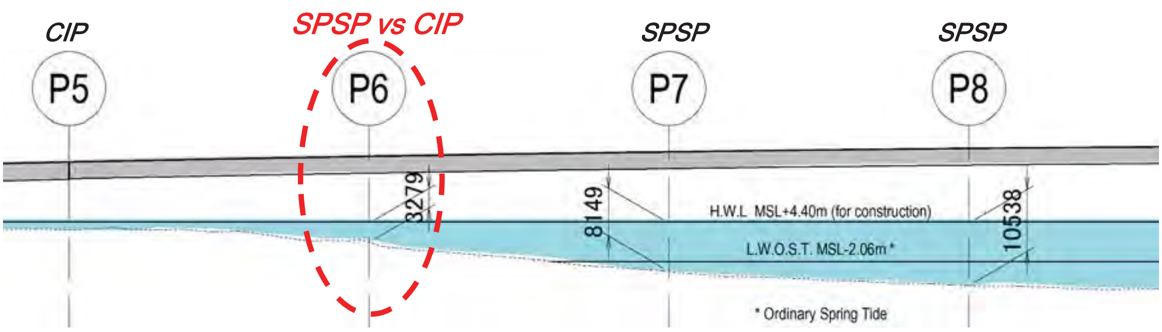
2.Basic Design Stage

(2) Foundation type at Riverfront Pier

At pears (P6,P23) located at a changing point from a flood channel to a low-flow channel, the riverbed elevation is much shallower than other pier locations.

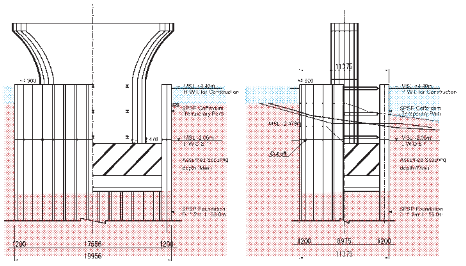
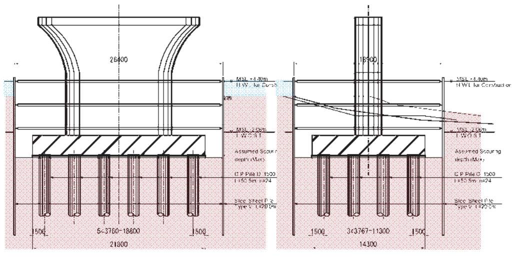
Thus, applicability of conventional cofferdam for a cost saving was performed.

- Alternative-1: SPSP Foundation-cum-cofferdam (adopted in Pre-Design)
- Alternative-2: CIP Pile Foundation with Steel Sheet Pile cofferdam



3.6 Selection of foundation and Substructure

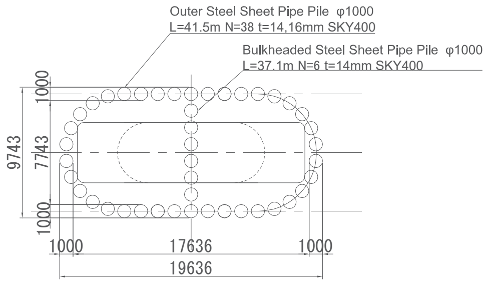
2.Basic Design Stage (2) Foundation type at Riverfront Pier

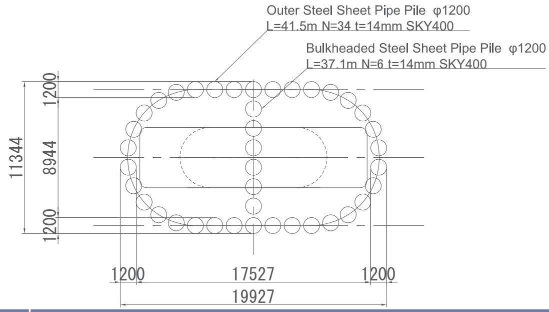
Evaluation Item	Alt-1 : SPSP Foundation (D=1.2m) [selected in FS]		Alt-2 : CIP Pile Foundation (D=1.5m)	
Schematic View	 D=1200mm x 44 nos. (L=55.0m)		 D=1500mm x 24 nos. (L=50.5m)	
Workability & Quality Control	- Sufficient water tightness to a planned water head - Changes of pile length during construction is available - Facile quality control due to use of prefabricated steel pipe piles - Careful adjustment is necessary for driving of deep steel pipes	◎	- Sufficient water tightness to a planned water head - Flexible to changes of pile length during construction - Careful quality control is necessary for in-situ concrete casting - Careful quality control is necessary for construction of deep borehole	○
Structural Aspect	- Sufficient to support a superstrucure reaction	○	- Sufficient to support a superstrucure reaction	○
Cost Ratio	1.87	△	1.00	◎
Construction Period	4.4 Months	○	4.1 Months	◎
Environmental Aspect	- Louder noise and larger vibration than CIP pile construction of foundation - Smaller amount of disposal of excavated soil	○	- Lower noise and vibration than SPSP foundation construction - Larger amount of disposal of excavated soil	○
Evaluation	Less Recommended		Recommended	

Legend : ◎ Very Good, ○ Good, △ Average

3.6 Selection of foundation and Substructure

2.Basic Design Stage (3) Diameter and thickness of SPSP

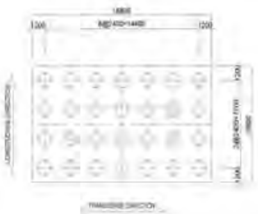
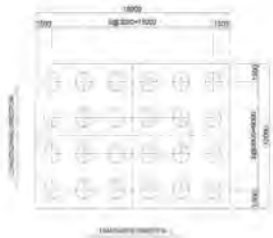
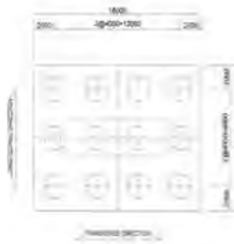




		Dia.1.0m	Dia.1.2m
Dimension Outer Pile Bulkhead Pile		19.6m x 9.7m Nos.38 x 29.0m (t=16mm) Nos.38 x 25.5m (t=14mm) Nos. 6 x 37.1m (t=14mm)	19.9m x 11.3m Nos.34 x 54.5m (t=14mm) Nos.6 x 37.1m (t=14mm)
Design Results	Displacement (cm) Bearing (kN/pile) Stress (N/mm^2)	3.9 < 5.0 (OK) 1726 < 5540 (OK) 186 < 210 (OK)	3.4 < 5.0 (OK) 1923 < 6575 (OK) 179 < 210 (OK)
Total Weight of Steel Pipes		8,161kN	8,320kN
Construction Cost/ Period		Higher cost/ Longer period	Lower cost/ Shorter period
Evaluation		Less Recommended	**Most Recommended** (although larger dimension and slightly heavier of steel pipes, construction period is prioritized for evaluation because SPSP is critical work.)

3.6 Selection of foundation and Substructure

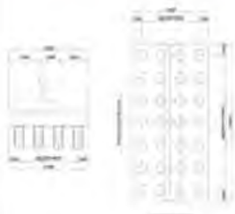
2.Basic Design Stage (4) Diameter of CIP Pile of Piers

Pile Diameter				Cast in Place RC Piles φ1.2m				Cast in Place RC Piles φ1.5m				Cast in Place RC Piles φ2.0m			
Outline Drawing															
Design Results	item	mark	unit	Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction	
				Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation
	Maximum Pile Reactions	Pmax	kN	1,608.0	3,089.1	1,608.0	2,978.8	2,003.8	3,540.6	2,003.8	3,582.1	3,895.5	6,853.1	3,895.5	7,390.5
		Ra	kN	3,530.0	5,516.0	3,530.0	5,516.0	4,632.0	7,294.0	4,632.0	7,294.0	6,523.0	10,403.0	6,523.0	10,403.0
		σ/σa	-	0.46	0.56	0.46	0.54	0.43	0.49	0.43	0.49	0.60	0.66	0.60	0.71
	Amount of Displacement	δx	mm	0.0	14.4	0.0	14.8	0.0	12.8	0.0	13.4	0.0	17.3	0.0	17.9
		δxa	mm	15.0	15.0	15.0	15.0	15.0	15.0	15.0	15.0	20.0	20.0	20.0	20.0
		R	-	0.00	0.96	0.00	0.98	0.00	0.85	0.00	0.89	0.00	0.86	0.00	0.89
	Stress of a Pile	σs	N/mm²	-17.0	211.5	-17.0	267.6	-14.0	184.5	-14.0	231.6	-15.5	180.5	-15.5	242.2
		σsa	N/mm²	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0
		σ/σa	-	0.09	0.71	0.09	0.89	0.07	0.61	0.07	0.77	0.08	0.60	0.08	0.81
	Maximum Stress of a Pile			σs= 268 kN/m² <σsa= 300kN/m² (OK)				σs= 232 kN/m2<σsa= 300kN/m2 (OK)				σs= 242 kN/m2<σsa= 300kN/m2 (OK)			
Constructibility				The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.			△	This alternative entails the smallest amount of pile works.			○	This alternative entails smaller amount of pile works.			◎
Construction Period				The amount of pile works including ground excavation is consirably smaller.			◎	The amount of pile works including ground excavation is consirably the smallest.			◎	The amount of pile works including ground excavation is consirably large.			◎
Environmental Aspect				This alternative entails the smallest amount of excavation works.			◎	This alternative entails small amount of excavation works.			△	This alternative entails the largest amount of excavation works.			○
Cost Ratio				1.237				1.235				1.000			◎
Judge												◎			

Note ◎:Good, ○:Fair, △:Not Recommended

3.6 Selection of foundation and Substructure

2.Basic Design Stage (5) Diameter of CIP Pile of Abutment

Pile Diameter				Cast in Place RC Piles φ1.2m		Cast in Place RC Piles φ1.5m		Cast in Place RC Piles φ2.0m	
Outline Drawing									
Design Results	item	mark	unit	Bridge's Longitudinal Direction		Bridge's Longitudinal Direction		Bridge's Longitudinal Direction	
				Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation	Presistent Situation	Seismic Situation
	Maximum Pile Reactions	Pmax	kN	1,208.0	1,844.6	1,605.1	2,400.7	2,404.1	3,761.0
		Ra	kN	2,797.0	4,400.0	3,730.0	5,916.0	5,353.0	8,603.0
		σ/σa	-	0.43	0.42	0.43	0.41	0.45	0.44
	Amount of Displacement	δx	mm	5.3	13.6	4.5	13.6	4.3	14.0
		δxa	mm	15.0	15.0	15.0	15.0	15.0	15.0
		R	-	0.35	0.90	0.30	0.91	0.29	0.94
	Stress of a Pile	σs	N/mm²	38.6	272.7	29.6	262.6	22.1	255.2
		σsa	N/mm²	160.0	300.0	160.0	300.0	160.0	300.0
		σ/σa	-	0.24	0.91	0.18	0.88	0.14	0.85
	Maximum Stress of a Pile			σs= 273 kN/m² <σsa = 300 kN/m² (OK)		σs= 263 kN/m² <σsa = 300 kN/m² (OK)		σs= 255 kN/m² <σsa = 300 kN/m² (OK)	
Constructability				The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.	△	This alternative entails the smaller amount of pile works.	○	This alternative entails the smallest amount of pile works.	◎
Construction Period				The amount of pile works including ground excavation is consirably smaller.	◎	The amount of pile works including ground excavation is consirably the smallest.	◎	The amount of pile works including ground excavation is consirably large.	◎
Environmental Aspect				This alternative entails the smallest amount of excavation works.	◎	This alternative entails small amount of excavation works.	○	This alternative entails the largest amount of excavation works.	△
Cost Ratio				1.095	○	1.000	◎	1.171	△
Overall Evaluation						◎			

Note ◎:Good, ○:Fair, △:Not Recommended

Detailed Design on Bago River Bridge Construction Project

Design Technology Transfer
Lecture : Substructure Design 2 “Cast-in-place RC Pile”

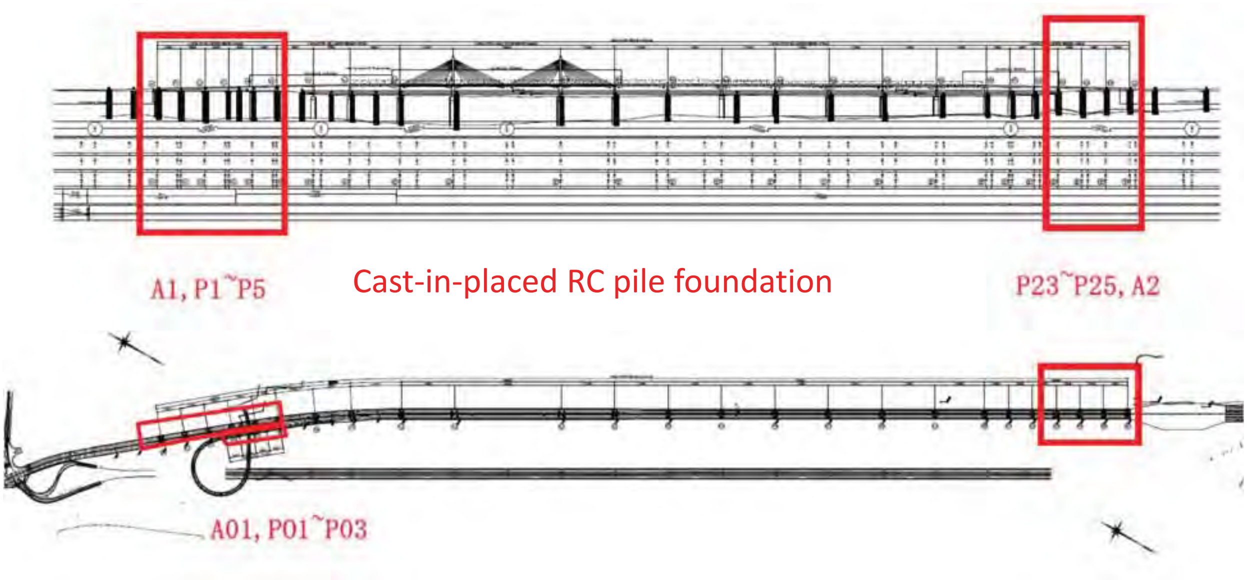
8th & 9th Nov 2017

Yasuhiro Takaoka (NIPPON KOEI)

Contents -Substructure Design “Cast-in-place RC Pile”-

Location Map

Bago River Bridge



A. Design Condition and General Arrangement

1. Design standard and Theory

The design is in accordance with SPECIFICATIONS FOR HIGHWAY BRIDGES (JSHB) published by JAPAN ROAD ASSOCIATION.

Foundation design is described in PART IV: substructures.

Also, other PARTs shall be referred as necessary.

- -PART I : COMMON
- -PART II : STEEO BRIDGES
- -PART III : CONCRETE BRIDGES
- -PART IV : SUBSTRUCTURES
- -PART V : SEISMIC DESIGN

1. Design standard and Theory

Point.1 :Requirements of the pile foundation

Pile foundations shall conform to the following requirements.

- The axial reaction at each pile head shall not exceed the allowable pile bearing capacity and allowable pull-out force.

  $P_{Nmax} \leqq R_a, \quad P_{Nmin} \leqq P_a$

- The displacement shall not exceed the allowable displacements.

  $\delta_f \leqq \delta_a$

- The stress generated in members of the pile foundations shall not exceed the allowable stress.

  $\sigma_s \leqq \sigma_{sa}, \quad \sigma_c \leqq \sigma_{ca}, \quad \tau \leqq \tau_a$

1. Design standard and Theory

Point.2 : Estimation of ultimate bearing capacity

- The ultimate bearing capacity shall be obtained either by the empirical bearing capacity estimation formula together with adequate geotechnical investigations, or from the results of vertical loading tests.

$$R_u = q_d\, A + U \sum L_i\, f_i$$

where,

R_u : ultimate bearing capacity of pile (kN)

A: area of pile tip (m^2)

q_d : ultimate end bearing capacity intensity per unit area (kN/m^2)

U: perimeter of pile (m)

L_i : thickness of soil layer considering shaft resistance (m)

f_i: maximum shaft resistance of soil layer considering pile shaft resistance (kN/m^2)

- The “ultimate end bearing capacity” and “ maximum shaft resistance intensity” for each pile installation method are given in JSHB, respectively, which have been obtained by analysis of numerous load tests by JSHB.

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1. Design standard and Theory

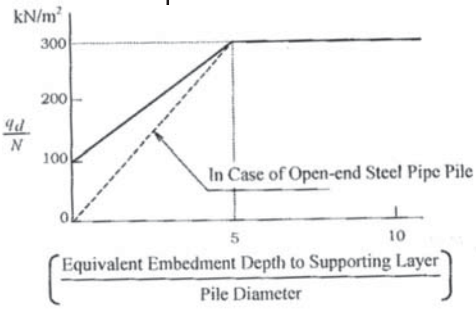
Ultimate end bearing capacity

・For Cast-in-placed RC pile

Ground Type	Ultimate Bearing Capacity End Bearing Intensity (kN/m^2)
Gravelly Layer and Sandy Layer ($N \geq 30$)	3,000
Sturdy Gravelly Layer ($N \geq 50$)	5,000
Hard Cohesive Soil Layer	$3q_u$

Notes) q_u : unconfined compressive strength (kN/m^2), N : N value from the Standard Penetration Test (SPT)

・For Driven pile



Maximum shaft resistance intensity

Pile Installation Method \ Ground Type	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-hammer Method)	$2N (\leq 100)$	c or $10N (\leq 150)$
Cast-in-place RC Pile Method	$5N (\leq 200)$	c or $10N (\leq 150)$
Bored Pile Method	$2N (\leq 100)$	$0.8c$ or $8N (\leq 100)$
Pre bored Pile Method	$5N (\leq 150)$	c or $10N (\leq 100)$
Steel Pipe Soil Cement Pile Method	$10N (\leq 200)$	c or $10N (\leq 200)$

(Note) c: cohesion of ground (kN/m^2), N: N value from SPT.

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1. Design standard and Theory

Point.3 : Pile reactions and displacements

-The pile reaction and displacement shall be calculated by representing the footing by a rigid structure, and the pile and ground <u>by a linear elastic structure</u> with the spring constants in the axial and lateral directions of the pile.

- Either of followings are applied for the calculation.

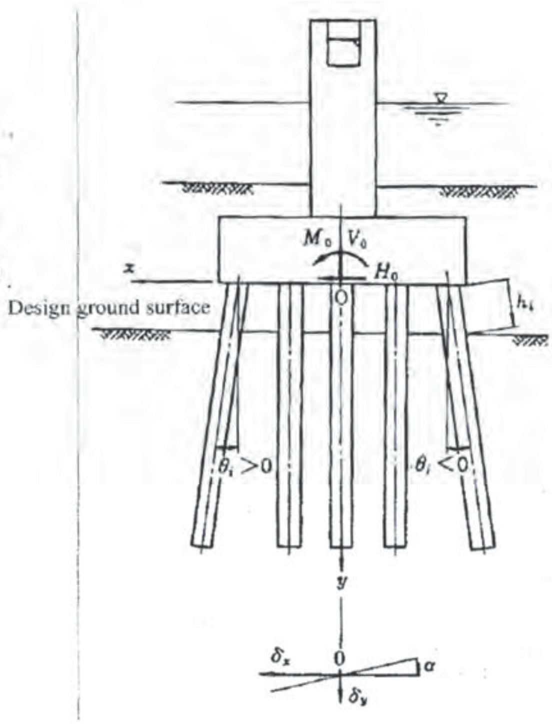
✓A method using a rigid-frame model in which a pile head is connected to a footing, making a pile a beam borne on a elastic floor

✓<u>A displacement method which solves a formula balancing displacement of the whole pile foundation by means of a spring matrix at a pile head.</u>

1. Design standard and Theory

Spring matrix

$$A_{xx} \bullet \delta_x + A_{xy} \bullet \delta_y + A_{x\alpha} \bullet \alpha = H_0$$
$$A_{yx} \bullet \delta_x + A_{yy} \bullet \delta_y + A_{y\alpha} \bullet \alpha = V_0$$
$$A_{\alpha x} \bullet \delta_x + A_{\alpha y} \bullet \delta_y + A_{\alpha\alpha} \bullet \alpha = M_0$$



1. Design standard and Theory

Point.4 : Sectional force on pile

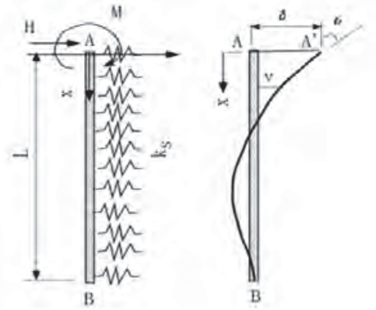
- The bending moments and shear forces in pile sections due to lateral forces and pile head moments shall be calculated by modeling the pile structure as a beam on an elastic foundation.

-If a coefficient of horizontal subgrade reaction is constant irrespective of depths and if an embedded depth of a pile is sufficiently long, the spring constants can be computed by using Hayashi-Chang Theory.

$$EI\frac{d^4 v}{dx^4} + k_s v = 0$$

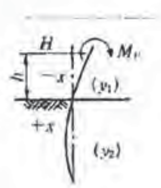
$$\frac{d^4 v}{dx^4} + 4\beta^4 v = 0$$

$$\beta = \sqrt[4]{\frac{k_H D}{4EI}}$$



1. Design standard and Theory

The calculation formula for a pile of semi-infinite length



Portion above the ground: $EI\frac{d^4y_1}{dx^4} = 0$

Portion in the ground: $EI\frac{d^4y_2}{dx^4} + p = 0$

$p = k_H D y_2$

H: lateral force of a pile (N)

M_t: moment as external force at the pile head (N·mm)

D: pile diameter (mm)

E: Young's modulus of the pile (N/mm²)

I: moment of inertia of the pile cross section (mm⁴)

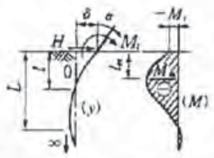
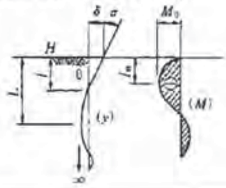
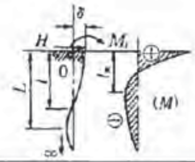
k_H: coefficient of horizontal subgrade reaction (N/mm³)

h: height above the ground surface where H and Mt act (mm)

$\beta = \sqrt[4]{k_H D / 4EI}\,(mm^{-1})$

$h_0 = \frac{M_t}{H}(mm)$

Pile embedded in the ground (h = 0)

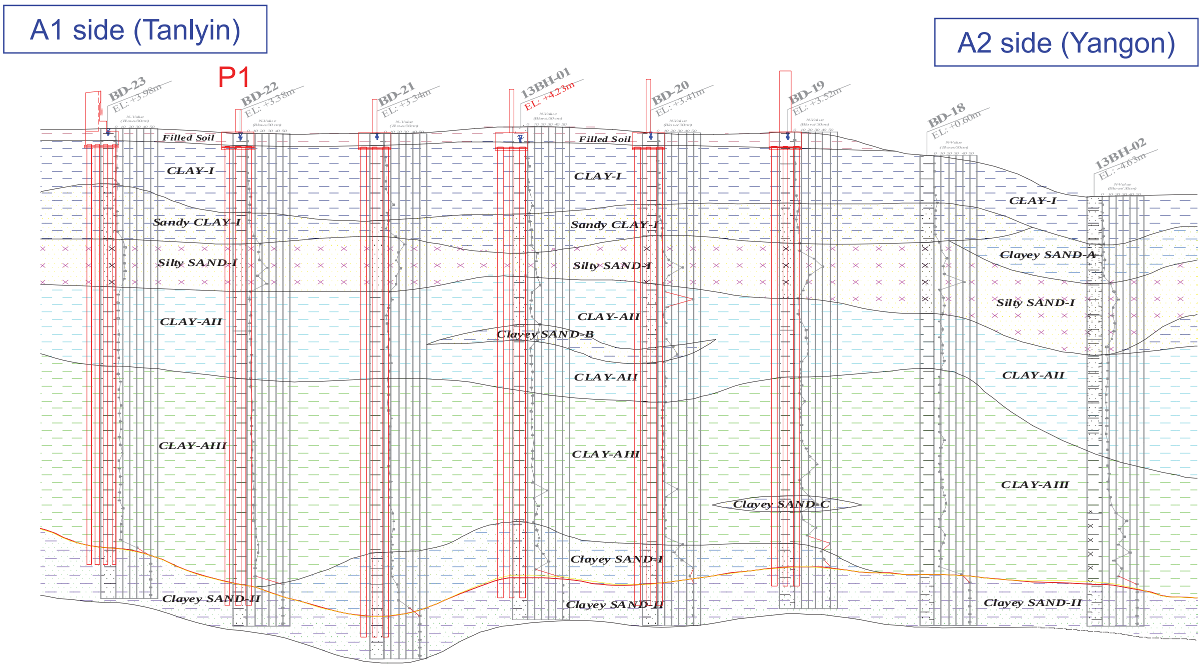
a) Basic system	b) When $M_t = 0$ (h0 = 0)	c) When the pile head does not rotate
		

2. Geological Condition

For the project, the geological condition along Bago river bridge have been investigated and evaluated at beginning.

- Standard Penetration Test has been conducted for all location of pier planned. Firmness of Clayey SAND-II indicated N-value 50 was confirmed, which is distributed uniformly at the elevation of around MSL-40.0~-60.0 m.
- Hence, Clayey SAND-II was selected as basement layer in the bridge design for this bridge site.

2. Geological Condition



2. Geological Condition

Each geotechnical parameters were obtained by laboratory etc. Specially, two parameters that have a profound effect for pile foundation are ;

1) Modulus of deformation of soils : E

⇒Obtained by pressure meter test

2) Reduction factor for due to liquefaction :D_E

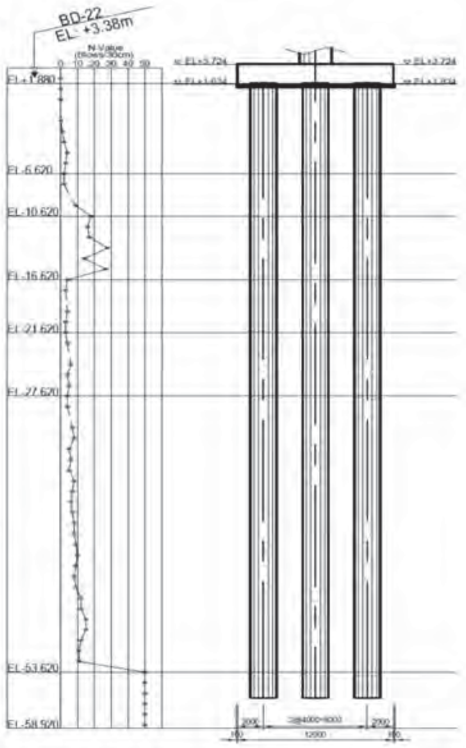
⇒ Determined by analysis of liquefaction possibility

<table>
<tr><td>Str. No</td><td colspan="3">A1</td><td colspan="3">P1</td><td colspan="3">P2</td></tr>
<tr><td>Br No.</td><td colspan="3">BD-23</td><td colspan="3">BD-22</td><td colspan="3">BD-21</td></tr>
<tr><td>Depth</td><td>Soil type</td><td>BD</td><td>DD</td><td>Soil type</td><td>BD</td><td>DD</td><td>Soil type</td><td>BD</td><td>DD</td></tr>
<tr><td>1</td><td rowspan="2">Filled Soil</td><td rowspan="2">N=2
E=1400
D_E=N/A</td><td rowspan="2">N=1
E=700
D_E=N/A</td><td rowspan="2">Filled Soil</td><td rowspan="2">N=2
E=1400
D_E=N/A</td><td rowspan="2">N=1
E=700
D_E=N/A</td><td rowspan="2">Filled Soil</td><td rowspan="2">N=2
E=1400
D_E=N/A</td><td rowspan="2">N=1
E=700
D_E=N/A</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td><td rowspan="5">CLAY-I</td><td rowspan="5">N=1
E=1800
D_E=N/A</td><td rowspan="5">N=1
E=900
D_E=N/A</td><td rowspan="7">CLAY-I</td><td rowspan="7">N=1
E=1800
D_E=N/A</td><td rowspan="5">N=1
E=900
D_E=N/A</td><td rowspan="8">CLAY-I</td><td rowspan="8">N=1
E=1800
D_E=N/A</td><td rowspan="5">N=1
E=900
D_E=N/A</td></tr>
<tr><td>4</td></tr>
<tr><td>5</td></tr>
<tr><td>6</td></tr>
<tr><td>7</td></tr>
<tr><td>8</td><td rowspan="6">Sandy CLAY-I</td><td rowspan="6">N=5
E=2500
D_E=N/A</td><td rowspan="3">N=3
E=2000
D_E=1/3</td><td rowspan="2">D_E=2/3</td><td rowspan="2">D_E=2/3</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td><td rowspan="4">Sandy CLAY-I</td><td rowspan="4">N=5
E=2500
D_E=N/A</td><td rowspan="4">N=3
E=2000
D_E=N/A</td><td>D_E=1</td></tr>
<tr><td>11</td><td rowspan="3">D_E=2/3</td><td rowspan="2">Sandy CLAY-I</td><td rowspan="2">N=5
E=2500
D_E=N/A</td><td rowspan="2">N=3
E=2000
D_E=1</td></tr>
<tr><td>12</td></tr>
<tr><td>13</td><td rowspan="8">Silty SAND-I</td><td rowspan="8">N=14
E=9800
D_E=1</td><td rowspan="8">N=15
E=6000
DE=1</td></tr>
<tr><td>14</td><td rowspan="7">Silty SAND-I</td><td rowspan="7">N=14
E=9800
D_E=1</td><td rowspan="7">N=15
E=6000
D_E=1</td><td rowspan="7">Silty SAND-I</td><td rowspan="7">N=14
E=9800
D_E=1</td><td rowspan="7">N=15
E=6000
D_E=1</td></tr>
<tr><td>15</td></tr>
<tr><td>16</td></tr>
<tr><td>17</td></tr>
<tr><td>18</td></tr>
<tr><td>19</td></tr>
<tr><td>20</td></tr>
</table>

2. Geological Condition

Soil Parameters

N	γt	Φ	Cu	E0	Layer Thickness	Soil Name
times	kN/m3	deg.	kN/m2	kN/m2	m	
1	18	0	6	700	1.5	F S
1	17.5	0	15	900	8.5	C-I
3	17.5	0	15	2000	4.0	S C-I
15	16.5	33.0	0	6000	6.0	S S-I
5	17.5	0	30	3200	5.0	C-AII
5	17.5	0	30	3200	6.0	C-AII
7	17.6	0	42	4900	26.0	C-AIII
50	19.0	35.0	0	35000	5.30	C S-II



3. General Arrangement of Pile

Pile length

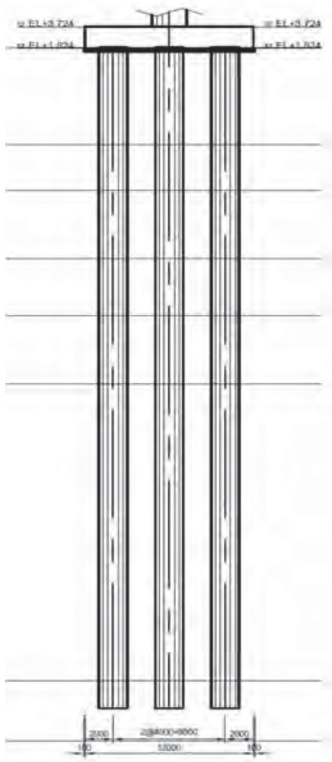
Pile length can be determined with followings.

1) Elevation of footing bottom

2) Embedment length for the footing : 100mm

3) Embedment length for the bearing layer of ground

: Around 1.0 D or more considering unevenness of bearing stratum



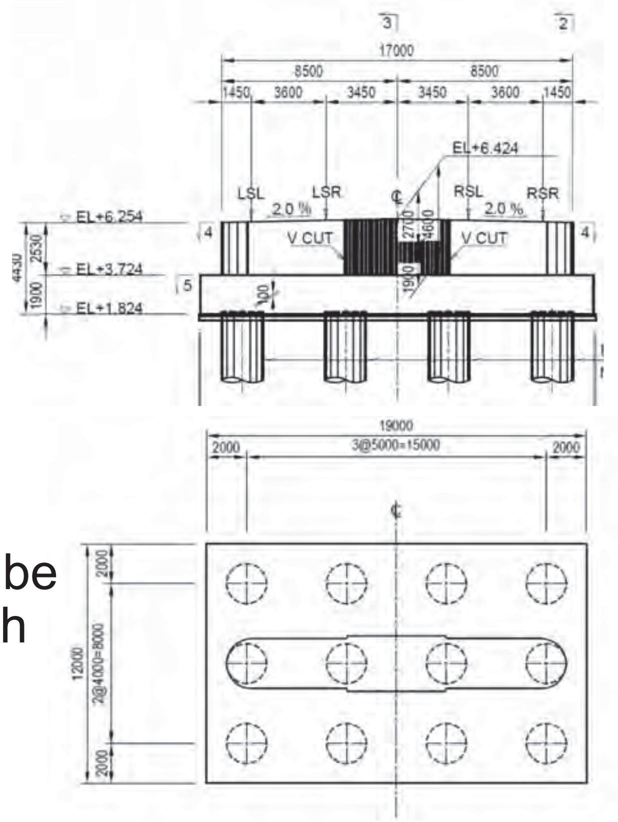
3. General Arrangement of Pile

Pile arrangement

・Piles can be arranged within footing dimension with following considerations.

- Pile Number
- Pile Diameter
- Pile Intervals

・Footing dimension shall be considered in relation with column dimension.

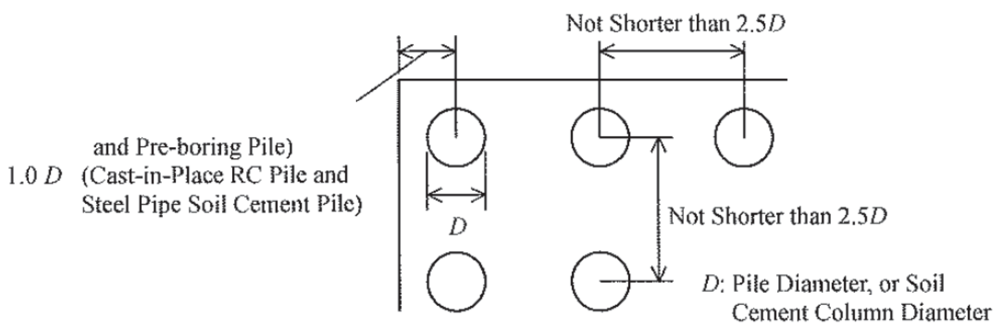


3. General Arrangement of Pile

Pile intervals

・The distance between adjacent pile centers of larger than 2.5times of the pile diameter are recommended without effect of group pile action.

・The distance between the outermost pile center and the footing edge can be equal to the pile diameter for cast-in place RC pile.

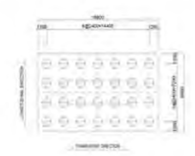
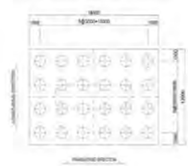


3. General Arrangement of Pile

Pile diameter

It is recommended to conduct comparison study to determine most economized Diameter of pile.

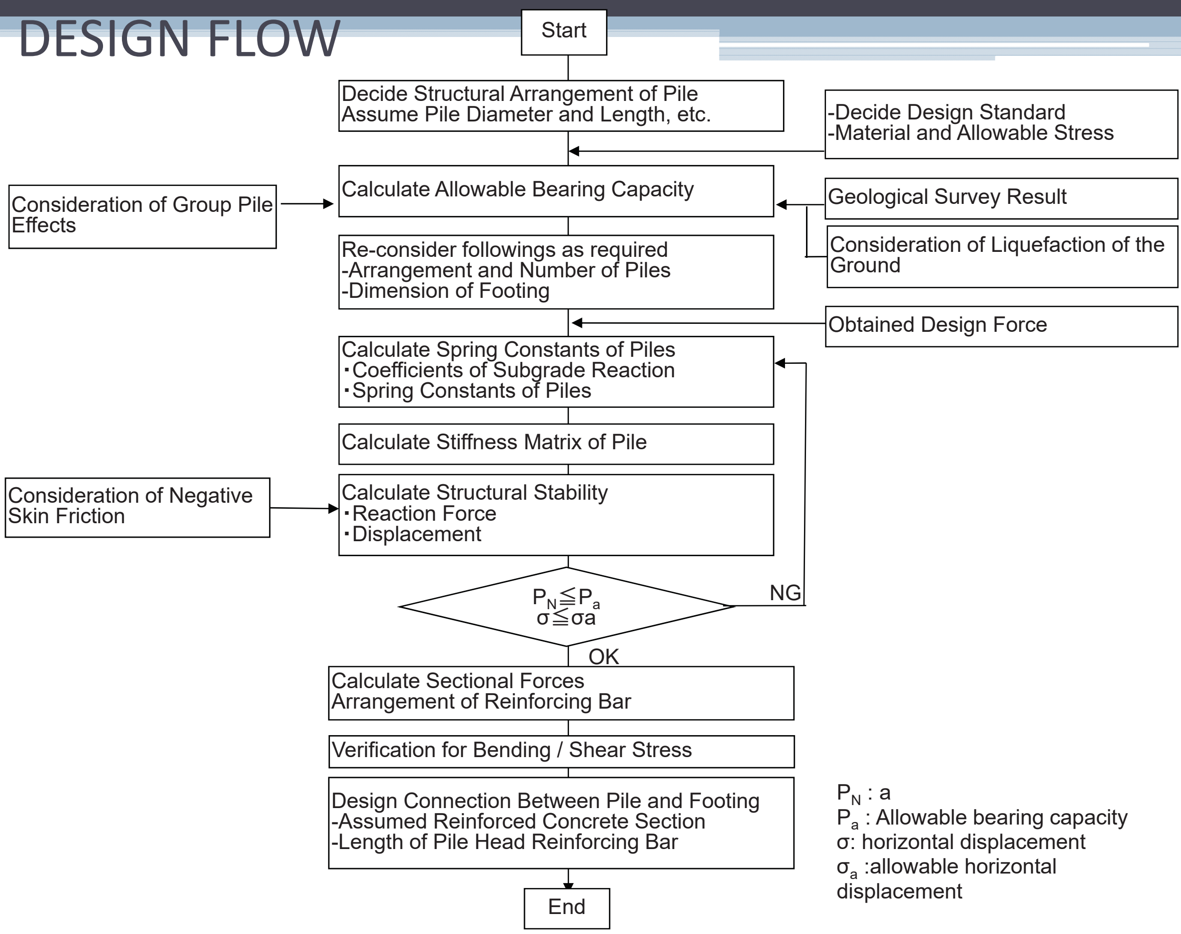
For the project, type of “D=2.0 m” was selected for on-land piers.

Pile Diameter			Cast in Place RC Piles φ1.2m				Cast in Place RC Piles φ1.5m				Cast in Place RC Piles φ2.0m			
Outline Drawing														
Design Results: item	mark	unit	Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction		Longitudinal Direction		Transverse Direction	
			Persistent Situation	Seismic Situation	Persistent Situation	Seismic Situation	Persistent Situation	Seismic Situation	Persistent Situation	Seismic Situation	Persistent Situation	Seismic Situation	Persistent Situation	Seismic Situation
Maximum Pile Reactions	Pmax	kN	1,608.0	3,089.1	1,608.0	2,978.8	2,003.8	3,540.6	2,003.8	3,582.1	3,895.5	6,853.1	3,895.5	7,390.5
	Ra	kN	3,530.0	5,516.0	3,530.0	5,516.0	4,632.0	7,294.0	4,632.0	7,294.0	6,523.0	10,403.0	6,523.0	10,403.0
	σ/σa	-	0.46	0.56	0.46	0.54	0.43	0.49	0.43	0.49	0.60	0.66	0.60	0.71
Amount of Displacement	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
Stress of a Pile	σs	N/mm²	-17.0	211.5	-17.0	267.6	-14.0	184.5	-14.0	231.6	-15.5	180.5	-15.5	242.2
	σsa	N/mm²	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0	-200.0	300.0
	σ/σa	-	0.09	0.71	0.09	0.89	0.07	0.61	0.07	0.77	0.08	0.60	0.08	0.81
Maximum Stress of a Pile			σs= 268 kN/m² < σsa= 300kN/m² (OK)				σs= 232 kN/m2 < σsa = 300kN/m2 (OK)				σs= 242 kN/m2 < σsa= 300kN/m2 (OK)			
Constructibility			The amount of number of pile is largest and thus this alternative is the most inferior one in terms of constructability.			△	This alternative entails the smallest amount of pile works.			[illegible]	This alternative entails smaller amount of pile works.			[illegible]
Construction Period			The amount of pile works including ground excavation is considerably smaller.			[illegible]	The amount of pile works including ground excavation is considerably the smallest.			[illegible]	The amount of pile works including ground excavation is considerably large.			[illegible]
Environmental Aspect			This alternative entails the smallest amount of excavation works			[illegible]	This alternative entails small amount of excavation works.			[illegible]	This alternative entails the largest amount of excavation works.			[illegible]
Cost Ratio			1.237				1.235				1.000			◎
Judge											◎			

Note: ◎ Good, ○ Fair, △ Not Recommended

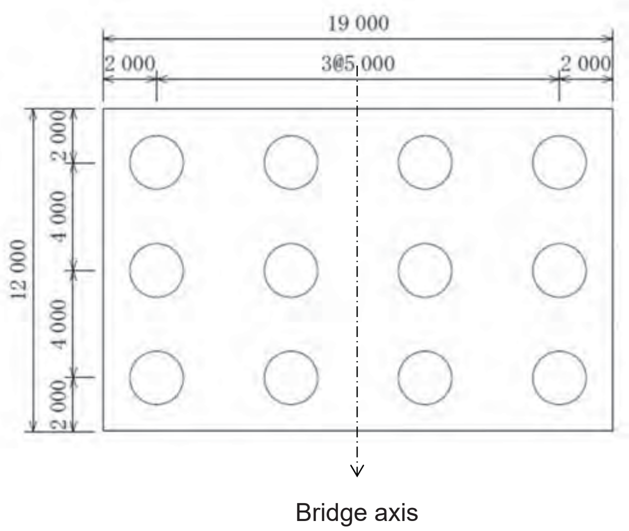
B. Design Calculation of Cast-in-placed RC Pile

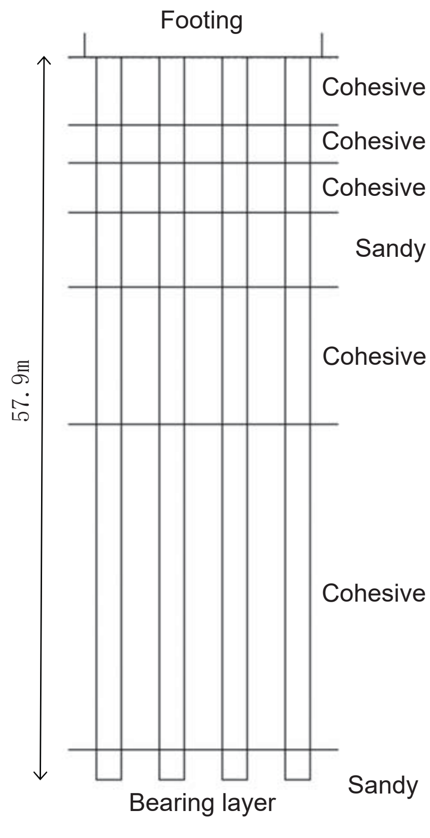
DESIGN FLOW



1. Structural Model

Structural model for design calculation of foundation





2. Structural Stability

2.1 ALLOWABLE BEARING CAPACITY

The axial allowable bearing capacity of a single pile can be obtained from equation below;

$$Ra = \frac{\gamma}{n} \cdot (Ru - Ws) + Ws - W$$

n : safety factor 3.0 (ordinary)

2.0 (seismic)

γ : modification of foefficient for factor of safety depending on ultimate bearing capacity estimation method = 1.0

Ru : ultimate bearing capacity of pile (kN)

$$Ru = qd \cdot Ap + U \cdot \Sigma(Li \cdot fi)$$

qd : ultimate end bearing capacity intensity per unit area (kN/m^2)

Ap : area of pile tip (m^2)

U : perimeter of pile (m)

Li : thickness of soil layer considering shaft resistance(m)

fi : maximum shaft resistance of soil layer considering pile shaft resistance (kN/m^2)

However, shaft friction is not considered in the range of 1.0・D from the tip of pile.

Ws : effective weight of soil replaced by pile(kN)

$Ws = Ap \cdot \Sigma(\gamma i \cdot Li)$

γi : effective unit weight of soil (kN/m^3)

W : effective weight of pile

End bearing capacity intensity
(For Cast-in place RC pile)

Ground Type	Ultimate Bearing Capacity End Bearing Intensity (kN/m^2)
Gravelly Layer and Sandy Layer ($N \geq 30$)	3,000
Sturdy Gravelly Layer ($N \geq 50$)	5,000
Hard Cohesive Soil Layer	$3q_u$

Notes) q_u : unconfined compressive strength (kN/m^2),
N : N value from the Standard Penetration Test (SPT)

(Source : JSHB)

Maximum shaft resistance intensity

Pile Installation Method \ Ground Type	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-hammer Method)	$2N$ (≤ 100)	c or $10N$ (≤ 150)
Cast-in-place RC Pile Method	$5N$ (≤ 200)	c or $10N$ (≤ 150)
Bored Pile Method	$2N$ (≤ 100)	$0.8c$ or $8N$ (≤ 100)
Pre bored Pile Method	$5N$ (≤ 150)	c or $10N$ (≤ 100)
Steel Pipe Soil Cement Pile Method	$10N$ (≤ 200)	c or $10N$ (≤ 200)

(Note) c: cohesion of ground (kN/m^2), N: N value from SPT.

(Source : JSHB)

Calculation Example

For ordinary case

Layer No	Soil	Average N-value	Cohesion (kN/m^2)	Layer thickness Li(m)	γi (kN/m^3)	Ws (kN)	fi (kN/m^2)	Li・fi (kN/m)
1	Cohesive	1.0	0.0	5.444	7.70	131.69	0.0	0.00
2	Cohesive	1.0	0.0	3.000	7.70	72.57	0.0	0.00
3	Cohesive	3.0	0.0	4.000	7.70	96.76	0.0	0.00
4	Sandy	15.0	0.0	6.000	7.70	145.14	75.0	450.00
6	Cohesive	5.0	30.0	11.000	7.70	266.09	30.0	330.00
7	Cohesive	7.0	42.0	26.000	7.80	637.12	42.0	1092.00
8	Sandy	50.0	0.0	0.456	10.20	14.61	200.0	91.20
Sum①								1963.20
8'	Sandy	50.0	0.0	2.000	10.20	64.09	200.0	400.00
Sum②				57.900		1428.07		2363.20

Calculation Example

- Calculation of each values

  qd= 3,000 (kN/m²)

  U = π・2.0000 = 6.283 (m)

  $$Ap = \frac{\pi}{4} \cdot 2.000^2 = 3.14159 \ (m^2)$$

- Ultimate bearing capacity of pile (Ordinary Case)

  Ru = qd・Ap ＋ U・Σ(Li・fi)

  = 3000・ 3.14159＋ 6.283・ 1963.2 = 21,760 (kN)

- Allowable bearing capacity of pile (Ordinary Case)

$$Ra = \frac{\gamma}{n} \cdot (Ru - Ws) + Ws - W$$

$$= \frac{1.0}{3.0} \cdot (21760 - 1428.1) + 1428.1 - 2673.9 = 5531 \ (kN)$$

2.2 Axial allowable pull-out force of pile

The axial allowable pull-out force of a single pile can be obtained from equation below;

$$Pa = \frac{1}{n} \cdot Pu + W$$

Pu : ultimate pull-out force of pile (kN)

$$Pu = U \cdot \Sigma (Li \cdot fi)$$

n : safety factor 6.0 (ordinary)

3.0 (seismic)

W : effective weight of pile

Moreover, It is recommended pull-out forces are not generated in ordinary load except limited case.

2.3 Design Force

For the foundation design, design force (V, H, M) worked at the center of footing bottom is required.

The design force can be obtained by design calculation of super-structure and sub-structure.

Image of transmitting design force (Not rigid frame bridge)

Structural calculation of super structure

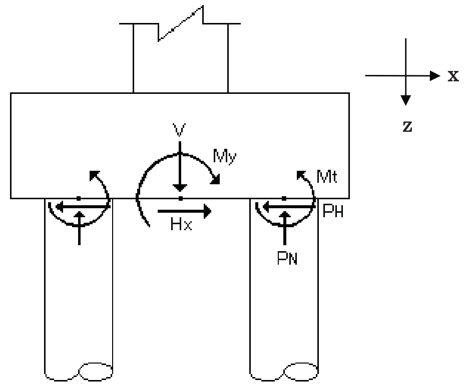
↓ By using force from super-structure

Structural calculation of sub-structure

↓ By using force from sub-structure

Structural calculation of foundation

Design force at footing bottom



2.3 Design Force

In foundation design, the stability and member stresses should be verified with consideration of the most adverse case of the load combination.

The design verification for the foundation of piers should be performed in both the longitudinal and transverse directions.

Example

	No	Load case
Longitudinal direction	1	Dead
	2	Dead + Live
	3	Dead + Live (Buoyancy)
	4	Dead + Live + Tempareture
	5	Dead + Live + Tempareture (Buoyancy)
	6	Seismic effect
	7	Seismic effect ((Bouyancy)

	No	Load case
Transverse direction	1	Dead
	2	Dead + Live
	3	Dead + Live (Buoyancy)
	4	Seismic effect
	5	Seismic effect ((Bouyancy)

2.4 Spring Constants of Pile

(1) Coefficient of Subgrade Reaction

The coefficient of subgrade reaction is defined as

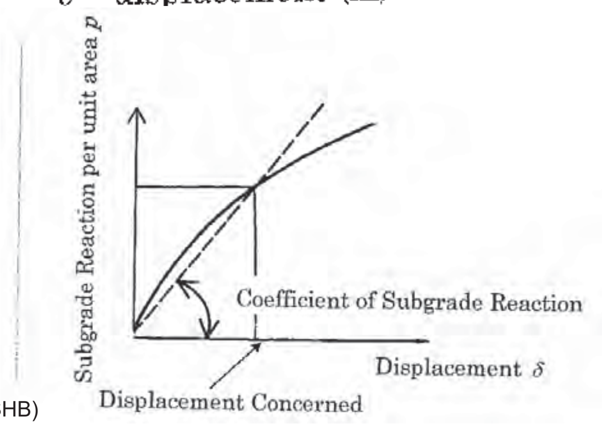
$$k = \frac{p}{\delta}$$

where,

k : coefficient of subgrade reaction (kN/m^3)

p : subgrade reaction per unit area (kN/m^2)

δ : displacement (m)



(Source : JSHB)

Displacement concerned can be considered as 1% of pile diameter

2.4 Spring Constants of Pile

(2) Coefficient of Horizontal Subgrade Reaction

A coefficient of Horizontal Subgrade Reaction is defined as subgrade reaction / displacement concerned at pile head. It shall be obtained by.

$$k_H = k_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

Where,

kH : coefficient of horizontal subgrade reaction obtained with regarding the value as the average from the design ground surface to the depth equal to 1/β(kN/m^3)

kHo : coefficient of horizontal subrgrade reaction, corresponding to the value obtained by the plate bearing test with a rigid disk of diameter 0.3m.

$$kHo = \frac{1}{0.3} \cdot \alpha \cdot Eo = \frac{1}{0.3} \cdot \frac{\Sigma(\alpha \cdot Eoi \cdot Li)}{1/\beta}$$

Li ：each layer depth in the range of 1/β (m)

BH ：equivalent loading width of foundation (m)

α ：coefficient for the estimation of the coeffient of subgrade reaction

Eo ：modulus of deformation of ground at the design location (kN/m²)

BH is obtained by using the averaged value of modulus of deformation of ground from the design ground surface to the depth 1/β as following equation.

$$BH = \sqrt{\frac{D}{\beta}}$$

Where,

D ：pile diameter (m)

1/β ：ground depth relating to the horizontal resistance (m)

β ：characteristic value of pile (m^{-1})

EI ：flexural stiffness of pile (kN/m²)

$$\beta = \sqrt[4]{\frac{kH \cdot D}{4 \cdot E \cdot I}}$$

Obtain of β by repeated calculation

The value β shall be obtained by repeated calculation till the calculation value is correspond to the tentative value.

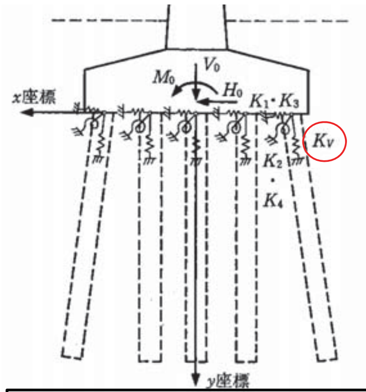
(Generally, the value of 1/β becomes 4 to 6 times of pile diameter.)

Calculation Example

Pile diameter	D = 2.0000	(m)
Young's modulus of pile	E = 2.50 × 10^7	(kN/m²)
Moment of inertia of pile	I = 0.785398164	(m⁴)
Characteristic value of pile β (tentative value)	0.1000	**(m^{-1})**
Ground depth relating to the horizontal resistance 1/β	11.8305	(m)
Average of α・Eo	4859.5	(kN/m²)
Equivalent loading width of pile BH	4.8643	(m)
Coefficient of horizontal subgrade reaction kHo	16198.4	(kN/m³)
〃 kH		(kN/m³)
Characteristic value of pile β（calculated value）	**0.084527**	**(m^{-1})**

(3) Axial Spring Constants of Pile

The axial spring constants of pile Kv is defined as the axial force capable of generating a unit displacement at the pile head in the longitudinal direction of the pile.



$$Kv = a \cdot \frac{Ap \cdot Ep}{L}$$

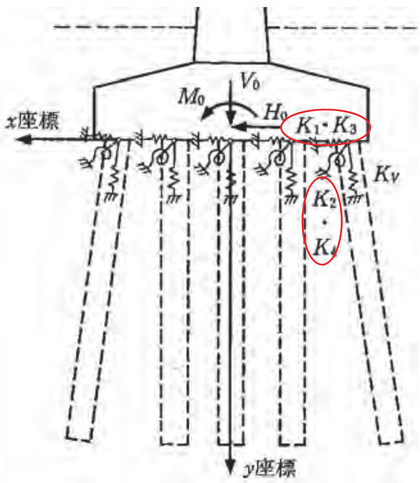
Estimated formula for Cast-in place RC pile, which is based on the result of load pressure test experienced at past.

$$a = 0.031 \cdot (L/D) - 0.15$$

Calculation Example
Ap : axial spring constant of pile = 3.14159 (m^2)
Ep : Young's modulus of pile = 2.50×10^7 (kN/m^2)
L : pile length = 57.9 (m), D : pile diameter = 2.0 (m)
a = 0.031・(L／D) − 0.15 = 0.7474
Kv = 1,013,896 (kN/m)

(4) Radial Spring Constant of Pile

Radial spring constants K1 to K4 of a pile are defined as below:

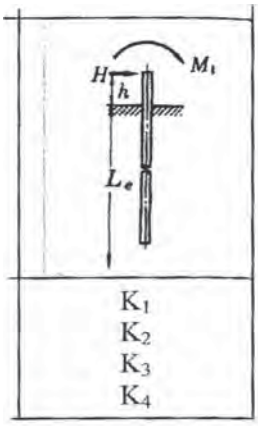


K1, K3 : radial force (kN/m) and bending moment (kN.m/m) to be applied on a pile head when displacing the head by a unit volume in a radial direction while keeping it from rotating.

K2, K4 : radial force (kN/rad) and bending moment (kN.m/rad) to be applied on a pile head when rotating the head by a unit volume while keeping it from moving in a radial direction.

(4) Radial Spring Constant of Pile

The constants can be computed by the Hayashi-Chang equation.



	Rigid frame of pile head		Hinged frame of pile head	
	h≠0	h=0	h≠0	h=0
K_1	$\frac{12EI\beta^3}{(1+\beta h)^3+2}$	$4EI\beta^3$	$\frac{3EI\beta^3}{(1+\beta h)^3+0.5}$	$2EI\beta^3$
K_2, K_3	$K_1 \cdot \frac{\lambda}{2}$	$2EI\beta^2$	0	0
K_4	$\frac{4EI\beta}{1+\beta h} \cdot \frac{(1+\beta h)^3+0.5}{(1+\beta h)^3+2}$	$2EI\beta$	0	0

Note) In case of pile length is a semi-infinite length ($\beta L \geqq 3$)

(Source : JSHB)

Where, β : characteristic value of a pile

λ : h + 1/β

kH: coefficient of horizontal subgrade reaction (kN/m3)

EI: bending rigidity of the pile (kN・m2)

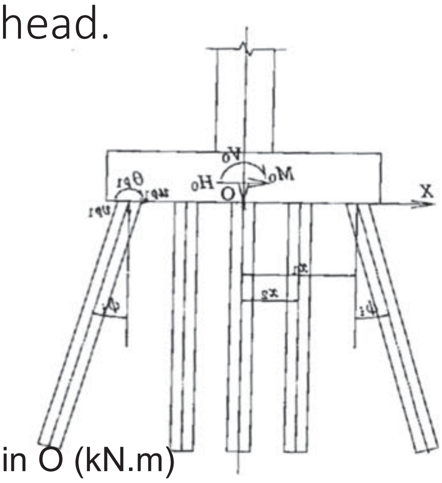
h: axial length of the pile above design ground surface (m)

2.5 Stiffness Matrix of Pile

- Reaction force of each piles can be obtained by "Displacement method" which is the relation between displacement and external force at footing bottom by assuming a footing as rigid body and by means of a spring matrix at a pile head.

Stiffness matrix

$$\begin{bmatrix} V \\ H \\ M \end{bmatrix} = \begin{bmatrix} Azz & Azx & Aza \\ Axz & Axx & Axa \\ Aaz & Aax & Aaa \end{bmatrix} \begin{bmatrix} \delta z \\ \delta x \\ \alpha \end{bmatrix}$$



V : vertical load acting at the origin O (kN)

H : lateral load acting at the origin O (kN)

M : moment of external force around the origin O (kN.m)

2.5 Stiffness Matrix of Pile

Stiffness matrix can be expressed as following equation.

$Azz \quad = \Sigma\ (Kv \cdot \cos^2\theta + K1 \cdot \sin^2\theta)\ i$

$Azx = Axz \quad = \Sigma\ (Kv \cdot \cos\theta \cdot \sin\theta - K1 \cdot \sin\theta \cdot \cos\theta)\ i$

$Aza = Aaz \quad = \Sigma\ (Kv \cdot X \cdot \cos^2\theta + K1 \cdot X \cdot \sin^2\theta + K2 \cdot \sin\theta)\ i$

$Axx \quad = \Sigma\ (Kv \cdot \sin^2\theta + K1 \cdot \cos^2\theta)\ i$

$Axa = Aax \quad = \Sigma\ (Kv \cdot X \cdot \sin\theta \cdot \cos\theta - K1 \cdot X \cdot \sin\theta \cdot \cos\theta - K2 \cdot \cos\theta)\ i$

$Aaa \quad = \Sigma\ \{Kv \cdot X^2 \cdot \cos^2\theta + K1 \cdot X^2 \cdot \sin^2\theta + (K2 + K3) \cdot X \cdot \sin\theta + K4\}\ i$

Note）”i” in the equation indicates i-th of pile.

Calculation Example

Ordinary case

$$\begin{bmatrix} Azz & Azx & Aza \\ Axz & Axx & Axa \\ Aaz & Aax & Aaa \end{bmatrix} = \begin{bmatrix} 12166752 & 0 & 0 \\ 0 & 571594 & -3922357 \\ 0 & -3922357 & 175738686 \end{bmatrix}$$

2.6 Reaction Force and Displacement

The pile axial force Pni, pile radial force Phi, and moment Mti acting on each pile head can be obtained with using displacement (δx, δz, α) at the footing origin obtained from the results of above-mentioned calculations, by following equations.

where, $Pni = Kv \cdot \delta zi'$

$Phi = K1 \cdot \delta xi' - K2 \cdot \alpha$

$Mti = -K3 \cdot \delta xi' + K4 \cdot \alpha$

$\delta zi = \quad (\delta z + \alpha \cdot Xi) \cdot \cos\theta i + \delta x \cdot \sin\theta i$

$\delta xi = -(\delta z + \alpha \cdot Xi) \cdot \sin\theta i + \delta x \cdot \cos\theta i$

Pile head vertical reaction Vi and horizontal reaction Hi are given by following equation.

$Vi = PNi \cdot \cos\theta i - PHi \cdot \sin\theta i$

$Hi = PNi \cdot \sin\theta i + PHi \cdot \cos\theta i$

Note）”i” in the equation indicates i-th of pile.

Calculation Example
-Direction : Longitudinal
-Load case : Dead + Live

- Force and displacement at the origin O

Acting force at the origin O
Vo = 43605.80 (kN)
Ho = 1030.00 (kN)
Mo = 4738.00 (kN.m)

Displacement at the origin O
δz = 3.58 (mm)
δx = 2.35 (mm)
α = 0.00007933 (rad)

- Reaction force of pile

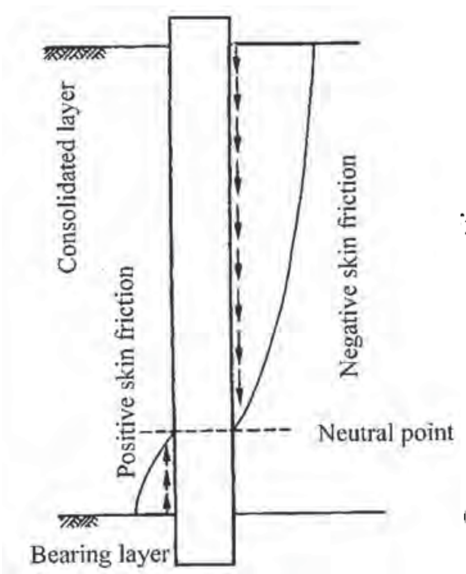
No	X(m)	Number	PN(kN)	PH(kN)	Mt(kN.m)	Vi(kN)	Hi(kN)	δfx(mm)
1	4.000	4	3955.54	85.83	-463.10	3955.54	85.83	2.35
2	0.000	4	3633.82	85.83	-463.10	3633.82	85.83	2.35
3	-4.000	4	3312.09	85.83	-463.10	3312.09	85.83	2.35

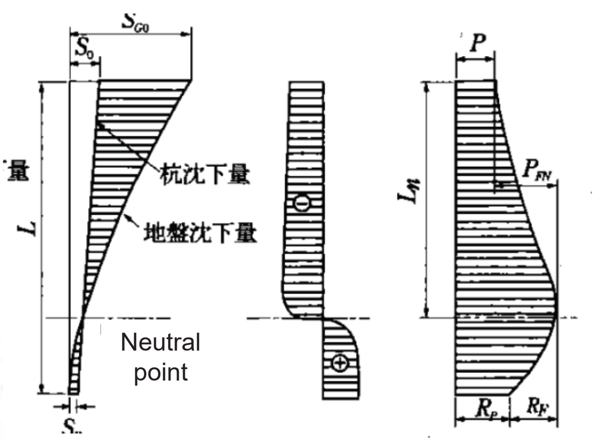
- Verification of axial force and displacement

PNmax = 3955.54 (kN) ≦ Ra = 5531.00 (kN) : OK
PNmin = 3312.09 (kN) ≧ Pa = 0.00 (kN) : OK
δf = 2.35 (mm) ≦ δa = 20.00 (mm) : OK

2.7 Consideration of Negative Skin Friction

In case of the consolidation settlement is likely to occur at the ground, the effects of negative skin friction shall be examined in to avoid failures and to maintain structural function.





(a) Settlement (b) Friction (c) axial force

2.7 Consideration of Negative Skin Friction

Reviewing bearing capacity with negative skin friction shall be conducted for dead load case, by;

$$R'_a = \frac{1}{1.5}\left(R'_u - W'_s\right) + W'_s - \left(R_{nf} + W\right)$$

-Ru': ultimate bearing capacity of a pile given by soil layers locating below the neutral point (kN)

-Rnf : negative skin friction (kN)

The location of a neutral point may be assumed to be at the lower end of the consolidated layer.

Calculation Example

Depth to neutral point : Lj = 12.444m

Reviewing vertical bearing capacity R ≦ Ra'

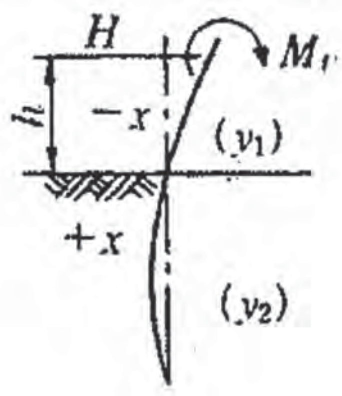
$$Ra' = \frac{Ru' - Ws'}{1.5} + Ws' - (Rnf + W) = 11035.6 \text{ (kN)}$$

	Dead load case
Bearing capacity check(kN)	3489 ≦ 11036 OK

3. Sectional Force

3.1 Sectional Force of pile

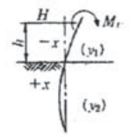
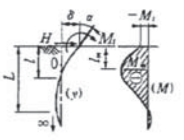
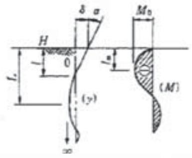
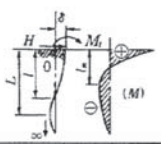
- The safety of pile sections against axial forces, bending moments, and shear forces shall be verified.
- The bending moments and shear force in pile sections due to lateral forces and pile head moments shall be calculated by modeling the pile structure as a beam on an elastic foundation.



3.1 Sectional Force of pile

- The bending moment shall be designed by using a lager value between bending moments with a pile head rigid connection and a pile head hinged one, even if the rigid connection is used.
- When a coefficient of horizontal subgrade reaction is uniform and if its embedded depth is 3/β or more, calculation may be carried out by assuming that <u>the pile is a beam of semi-infinite length with a constant coefficient of horizontal subgrade reaction</u>.

The calculation formula for a pile of semi-infinite length

Differential equations of deflection curves			Portion above the ground: $EI\frac{d^4y_1}{dx^4}=0$ Portion in the ground: $EI\frac{d^4y_2}{dx^4}+p=0$ $p=k_HDy_2$ *H*: lateral force of a pile (N) M_t: moment as external force at the pile head (N·mm) *D*: pile diameter (mm) *E*: Young's modulus of the pile (N/mm^2)	*I*: moment of inertia of the pile cross section (mm^4) k_H: coefficient of horizontal subgrade reaction (N/mm^3) *h*: height above the ground surface where H and Mt act (mm) $\beta=\sqrt[4]{k_HD/4EI}\left(mm^{-1}\right)$ $h_0=\frac{M_t}{H}(mm)$
State of a pile		Pile embedded in the ground (h = 0)		
Deflection curve and bending moment diagram		a) Basic system 	b) When $M_t=0$ (h0 = 0) 	c) When the pile head does not rotate 
a	Deflection curve, *y* (mm)	$y=\frac{H}{2EI\beta^3}e^{-\beta_x}\left[(1+\beta h_0)\cos\beta_x-\beta h_0\sin\beta_x\right]$	$y=\frac{H}{2EI\beta^3}e^{-\beta_x}\cos\beta_x$	$y=\frac{H}{4EI\beta^2}e^{-\beta_x}\left[\cos\beta_x+\sin\beta_x\right]$
b	Displacement of pile head *δ* (mm)	$\delta=\frac{H}{2EI\beta^3}+\frac{M_t}{2EI\beta^2}=\frac{1+\beta H_0}{2EI\beta^3}H$	$\delta=\frac{H}{2EI\beta^3}$	$\delta=\frac{H}{4EI\beta^3}=\frac{\beta H}{k_HD}$
c	Displacement of ground surface *f* (mm)	$f=\delta$	$f=\delta$	$f=\delta$

(Source : JSHB)

The calculation formula for a pile of semi-infinite length

d	Pile head inclination angle, α (rad)	$\alpha = \frac{H}{2EI\beta^2} + \frac{M_t}{EI\beta} = \frac{1+2\beta h_0}{2EI\beta^3} H$	$\alpha = \frac{H}{2EI\beta^2}$	$\alpha = 0$
e	Bending moment at each portion of the pile M (N·mm)	$M = -\frac{H}{\beta} e^{-\beta x} [\beta h_0 \cdot \cos \beta_x + (1+\beta h_0) \sin \beta_x]$	$M = -\frac{H}{\beta} e^{-\beta x} \sin \beta_x$	$M = -\frac{H}{2\beta} e^{-\beta x} (\sin \beta_x - \cos \beta_x)$
f	Shear force at each portion of the pile S (N)	$S = -He^{-\beta x} [\cos \beta_x - (1+2\beta h_0) \sin \beta_x]$	$S = -He^{-\beta x} (\cos \beta_x - \sin \beta_x)$	$S = -He^{-\beta x} \cos \beta_x$
g	Bending moment at pile head M_0 (N·mm)	$M_0 = -M_t = -Hh_0$	$M_0 = 0$	$M_0 = \frac{H}{2\beta}$
h	Bending moment at a point along an underground portion l_m M_m (N·mm)	$M_m = -\frac{H}{2\beta} \sqrt{(1+2\beta h_0)^2 + 1} \exp(-\beta l_m)$	$M_m = -\frac{H}{\beta} e^{-\frac{\pi}{4}} \cdot \sin \frac{\pi}{4} = -0.3224 \frac{H}{\beta}$	$M_m = -\frac{H}{2\beta} e^{-\frac{\pi}{2}} = -0.2079 M_0$
i	l_m (mm)	$l_m = \frac{1}{\beta} \tan^{-1} \frac{1}{1+2\beta h_0}$	$l_m = \frac{\pi}{4\beta}$	$l_m = \frac{\pi}{2\beta}$
j	Depth of a primary immobile point, l (mm)	$l = \frac{1}{\beta} \tan^{-1} \frac{1+\beta h_0}{\beta h_0}$	$l = \frac{\pi}{2\beta}$	$l = \frac{3\pi}{4\beta}$
k	Depth causing deflection angle zero, L (mm)	$L = \frac{1}{\beta} \tan^{-1} [-(1+2\beta h_0)]$	$L = \frac{3\pi}{4\beta}$	$L = \frac{\pi}{\beta}$

(Source : JSHB)

Calculation Example

- Sectional forces

	Pile head RIGID connection			Pile head HINGED connection		
Acting force at pile head H (kN) M (kN. m)	85. 83 -463. 10			85. 83 0. 00		
Radial spring constants K1 (kN/m) K2 (kN/rad) K3 (kN. m/m) K4 (kN. m/rad)	47633 326863 326863 3830000			19737 0 0 0		
Mt (kN. m) Mmax (kN. m) Z (m) 1/2Mmax (kN. m) S (kN) Z (m)	-463. 10 224. 28 14. 916 231. 55 68. 12 3. 017			0. 00 434. 78 11. 443 231. 55 -30. 36 20. 209		
Z (m)	δ x (mm)	M (kN. m)	S (kN)	δ x (mm)	M (kN. m)	S (kN)
0. 000	2. 346	-463. 10	85. 83	4. 349	0. 00	85. 83
0. 500	2. 304	-420. 97	82. 72	4. 163	41. 48	80. 14
1. 000	2. 256	-380. 37	79. 68	3. 978	80. 18	74. 70
1. 500	2. 203	-341. 28	76. 69	3. 794	116. 23	69. 50
2. 000	2. 146	-303. 66	73. 79	3. 612	149. 73	64. 55
2. 500	2. 085	-267. 48	70. 96	3. 431	180. 82	59. 85
3. 000	2. 021	-232. 69	68. 21	3. 253	209. 62	55. 38
3. 500	1. 954	-199. 25	65. 55	3. 078	236. 24	51. 15
4. 000	1. 884	-167. 12	62. 99	2. 905	260. 80	47. 15
4. 500	1. 812	-136. 25	60. 52	2. 736	283. 42	43. 38

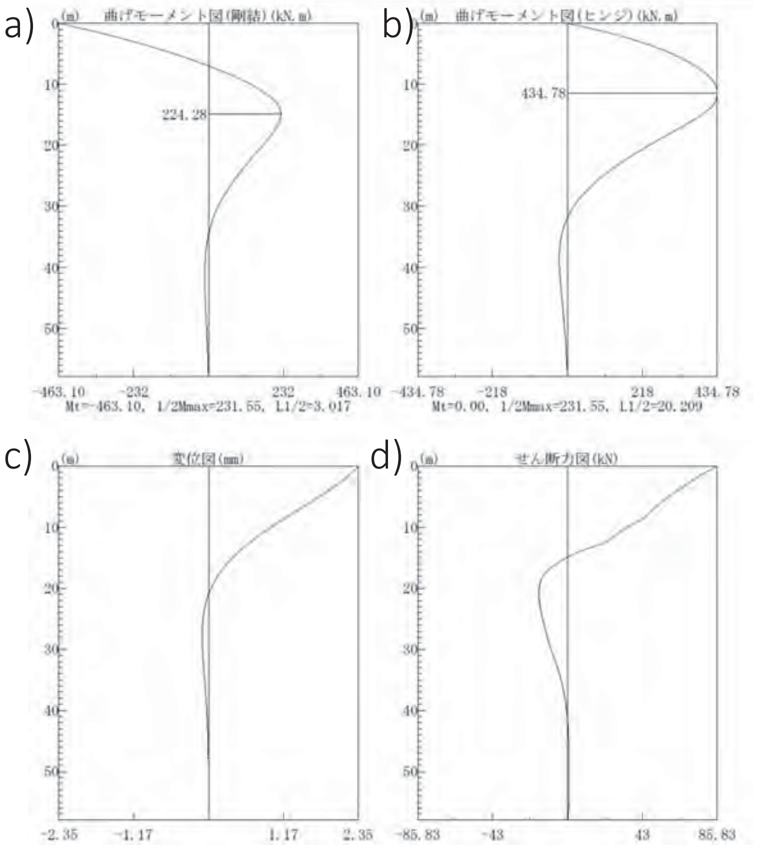
- Diagrams

a)Bending moment (Rigid)

b)Bending moment (hinged)

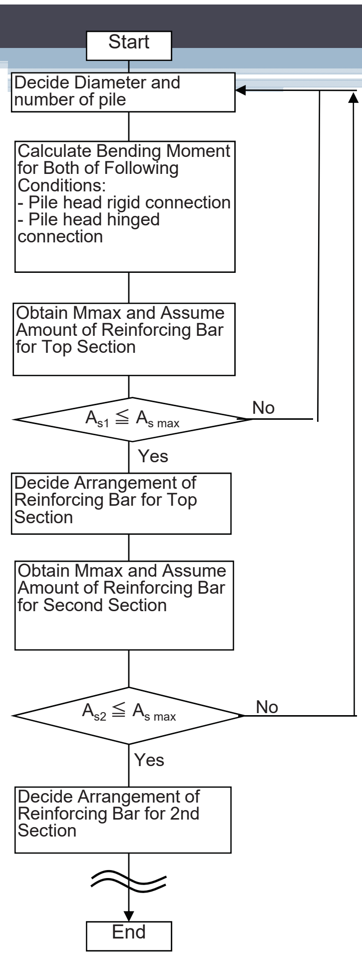
c)Displacement

d)Shear force



3.2 Arrangement of Reinforcing bar

- In general, the largest bending moment occurs at upper portion of the pile.
- In order to arrange re-bars in effective, the arrangement is designed for several sections depending the depth.

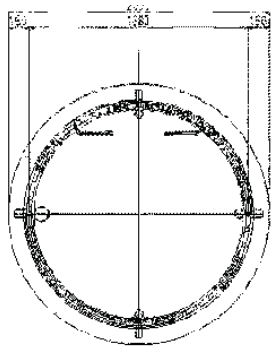


Example

In case if the section for re-bar arrangement is divided into three section.

1st Section

Low	Reinforcing bar	Concrete cover (mm)	As(cm^2)	ΣAs(cm^2)
1	D32- 44(@ 120)	160.0	349.448	349.448



2nd Section

Low	Reinforcing bar	Concrete cover (mm)	As(cm^2)	ΣAs(cm^2)
1	D32- 22(@ 240)	160.0	174.724	174.724

3rd Section

Low	Reinforcing bar	Concrete cover (mm)	As(cm^2)	ΣAs(cm^2)
1	D29- 22(@ 240)	160.0	141.328	141.328

4. Stress Verification of piles

4.1 Material and Allowable Stress

The allowable stresses shall be obtained by multiplying respective increase coefficients stipulated in each load cases respectively.

-Design Strength of Concrete : σ=24N/mm2

No	Load case	Increase coefficient	Allowable bending compressive stress σca	Allowable shear stress	
				τa1	τa2
1	Dead	1.00	8.00	0.230	1.700
2	Dead + Live	1.00	8.00	0.230	1.700
3	Dead + Live + Temperature	1.15	9.20	0.264	1.955
4	Seismic Effect	1.50	12.00	0.350	2.550

-Reinforcing bar　Material type ： SD345

No	Load case	Increase coefficient	Allowable bending compressive stress σsa’	Allowable bending tensile stress σsa
1	Dead	1.00	200.00	160.00
2	Dead + Live	1.00	200.00	160.00
3	Dead + Live + Temperature	1.15	230.00	184.00
4	Seismic Effect	1.50	300.00	300.00

4.2 Verification for Bending Stress

For the verification of RC members subjected to bending moments or axial force, it shall be verified that the stresses in the concrete and reinforcement of RC members calculated to comply with the following assumptions are smaller than the allowable stresses.

i) Fiber strains are proportional to the distance from the neutral axis.

ii) The tensile strength of the concrete is neglected

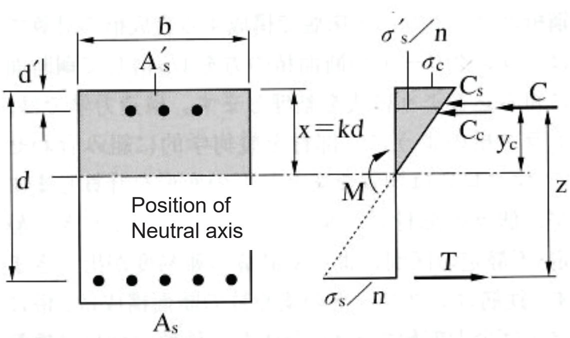
iii) The ratio of Young's modulus of the steel reinforcement to that of the concrete is 15.

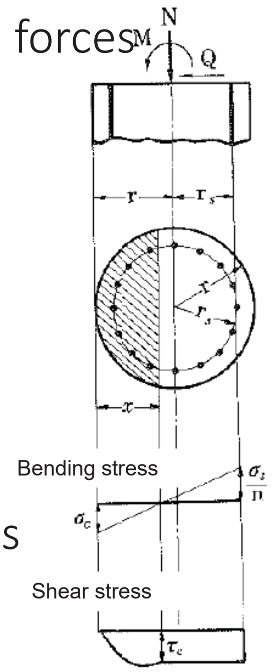
4.2 Verification for Bending Stress

General theory of RC section calculation is based on;

-Compatibility of strains between concrete and re-bars

-Equilibrium of Compressive &Tension forces





By RC section calculation, Bending stress on concrete and re-bars are obtained.

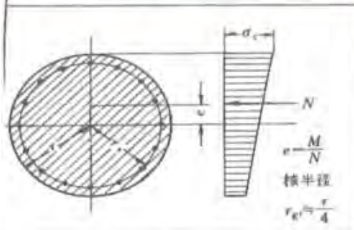
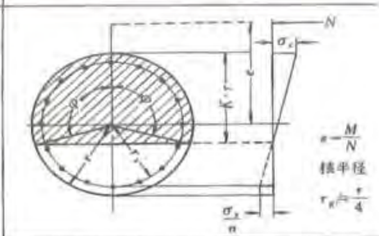
4.2 Verification for Bending Stress

For verification of bending Stress, following needs to be checked;

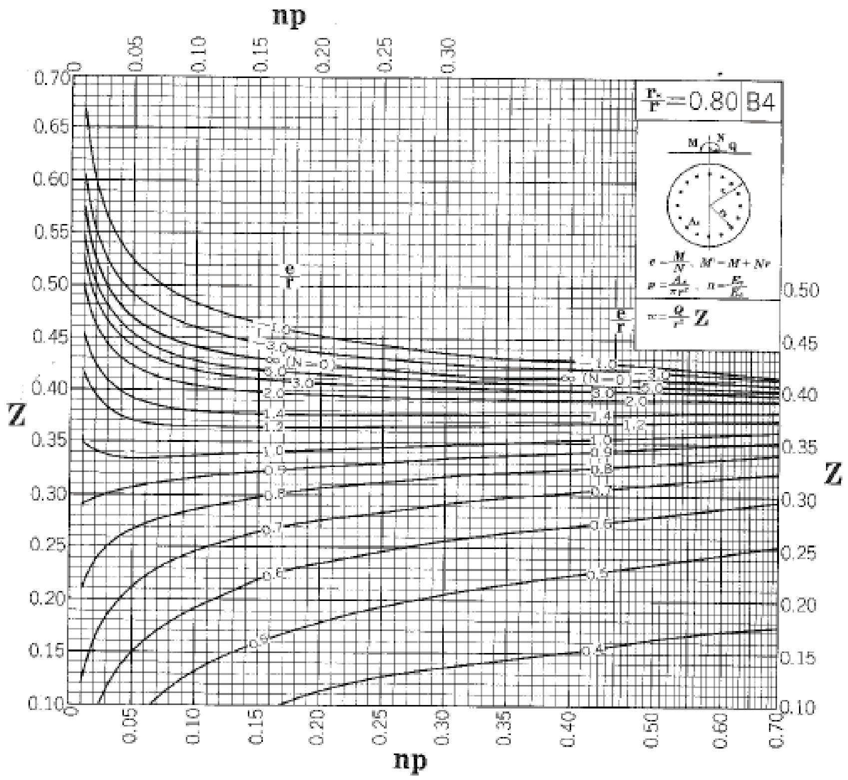
Compressive stress : σc ≦Allowable compressive stress σsa
Tensile stress : σs≦ Allowable tensile stress σsa

- The calculation of the working stresses σc and σs can be performed by computation with structural design software in practical.
- With use of formula or schematics, the calculation of RC section for simple RC section can be also achieved by hand.

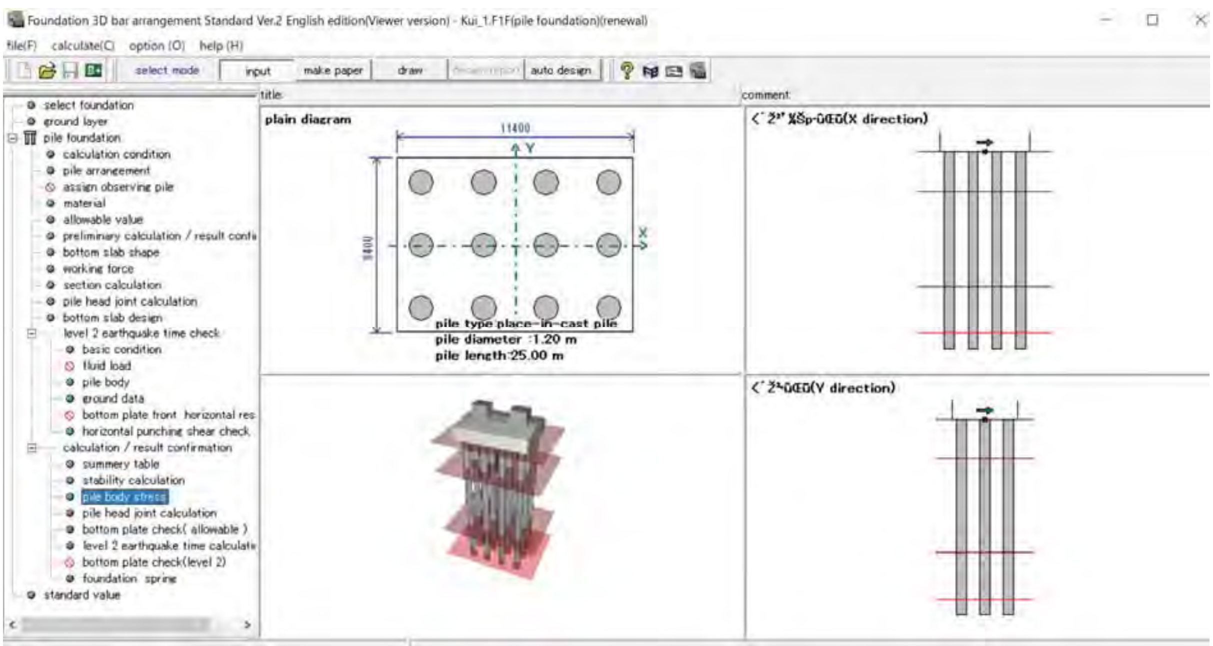
The formula for calculation of stresses on circular section

円形断面（eが核半径より小さい場合）	円形断面（eが核半径より大きい場合）
	
$P=\frac{A_s}{\pi r^2}$	$P=\frac{A_s}{\pi r^2}$
	$\frac{e}{r}=\frac{\frac{\varphi}{4}-\left(\frac{5}{12}-\frac{1}{6}\cos^2\varphi\right)\sin\varphi\cos\varphi+\frac{n\pi p}{2}\left(\frac{r_s}{r}\right)^2}{\frac{\sin\varphi}{3}(2+\cos^2\varphi)-\varphi\cos\varphi-n\pi p\cos\varphi}$
$C=\frac{1}{1+np}+\frac{e}{r}\cdot\frac{4}{(1+2np(r_s/r)^2)}$	$C=\frac{\frac{\sin\varphi}{3}(2+\cos^2\varphi)-\varphi\cos\varphi-n\varphi p\cos\varphi}{1-\cos\varphi}$
$\sigma_c=\frac{N}{\pi r^2}\cdot C$	$\sigma_c=\frac{N}{r^2 C}$ $\sigma_s=n\frac{r_s/r+\cos\varphi}{1-\cos\varphi}\sigma_c$

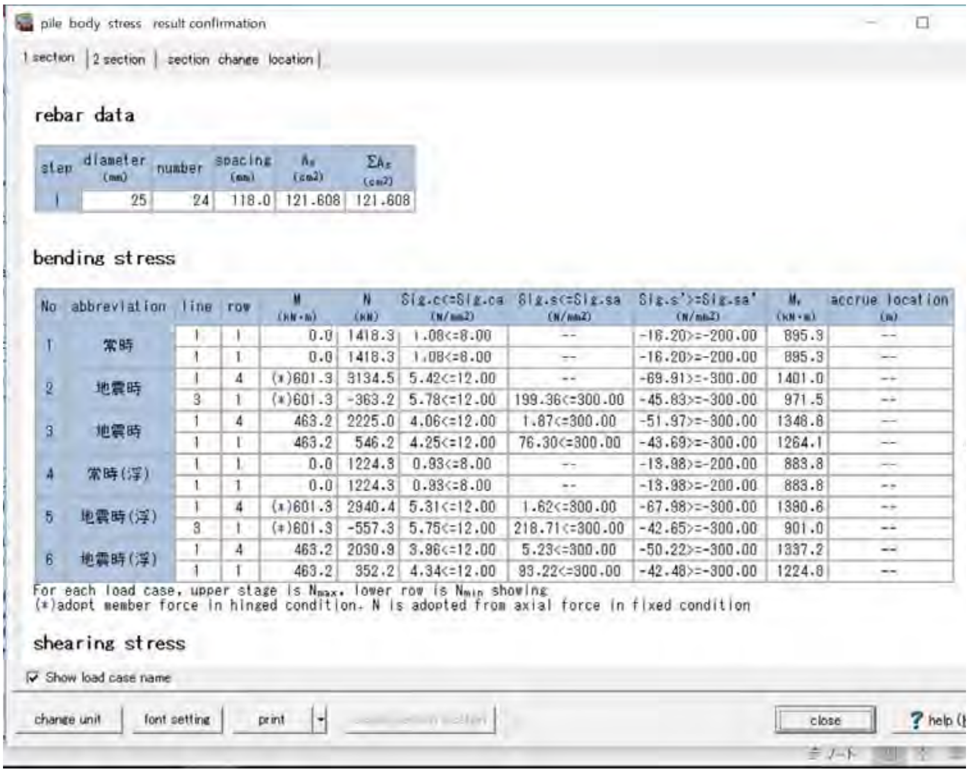
The schematic for calculation of stresses on circular section



Foundation calculation by structural software (FORUM8)



Foundation calculation by structural software (FORUM8)



pile body stress result confirmation

1 section | 2 section | section change location |

rebar data

step	diameter (mm)	number	spacing (mm)	A_s (cm2)	ΣA_s (cm2)
1	25	24	118.0	121.608	121.608

bending stress

No	abbreviation	line	row	M (kN·m)	N (kN)	Sig.c<=Sig.ca (N/mm2)	Sig.s<=Sig.sa (N/mm2)	Sig.s'>=Sig.sa' (N/mm2)	M_y (kN·m)	accrue location (m)
1	常時	1	1	0.0	1418.3	1.08<=8.00	--	-16.20>=-200.00	895.3	--
		1	1	0.0	1418.3	1.08<=8.00	--	-16.20>=-200.00	895.3	--
2	地震時	1	4	(*)601.3	3134.5	5.42<=12.00	--	-69.91>=-300.00	1401.0	--
		3	1	(*)601.3	-363.2	5.78<=12.00	199.36<=300.00	-45.83>=-300.00	971.5	--
3	地震時	1	4	463.2	2225.0	4.06<=12.00	1.87<=300.00	-51.97>=-300.00	1348.8	--
		1	1	463.2	546.2	4.25<=12.00	76.30<=300.00	-43.69>=-300.00	1264.1	--
4	常時(浮)	1	1	0.0	1224.3	0.93<=8.00	--	-13.98>=-200.00	883.8	--
		1	1	0.0	1224.3	0.93<=8.00	--	-13.98>=-200.00	883.8	--
5	地震時(浮)	1	4	(*)601.3	2940.4	5.31<=12.00	1.62<=300.00	-67.98>=-300.00	1390.6	--
		3	1	(*)601.3	-557.3	5.75<=12.00	218.71<=300.00	-42.65>=-300.00	901.0	--
6	地震時(浮)	1	4	463.2	2030.9	3.96<=12.00	5.23<=300.00	-50.22>=-300.00	1337.2	--
		1	1	463.2	352.2	4.34<=12.00	93.22<=300.00	-42.48>=-300.00	1224.8	--

For each load case, upper stage is N_{max}, lower row is N_{min} showing
(*)adopt member force in hinged condition. N is adopted from axial force in fixed condition

shearing stress

Show load case name

change unit | font setting | print | close | help

4.3 Verification for Shear Stress

1)When only the concrete carries the shear forces, the mean shear stresses τm calculated from 3) shall be smaller than the allowable shear stresses τa1.

2)When the concrete and diagonal tension reinforcement jointly carry the shear forces, the mean shear stresses τm shall exceed the allowable shear stresses τa2.

3)The mean shear stress of the concrete generated in a section of an RC member shall be calculated by ;

$$\tau_m = \frac{S_h}{b\,d}$$

τ_m : mean concrete shear stress generated in a member section (N/mm²)

S_h : shear force considering the variation in effective depth of the member (N), calculated from $S_h = S - \frac{M}{d}(\tan\beta + \tan\gamma)$

4.3 Verification for Shear Stress

When only the concrete bears the shear forces, the allowable shear stress τa1 should be modified in consideration of the following effects;

- Modification coefficient "Ce" for the effective depth d, at the member section
- Modification coefficient "Cpt" for the longitudinal tension bars ratio pt
- Modification coefficient "CN" for the axial compressive forces

Effective Depth d (mm)	Less than 300	1,000	3,000	5,000	More than 10,000
c_e	1.4	1.0	0.7	0.6	0.5

Longitudinal Tensile Reinforcement Ratio p_t (%)	0.1	0.2	0.5	0.5	1.0 or more
c_{pt}	0.7	0.9	1.0	1.2	1.5

$$CN = 1 + \frac{Mo}{M} \quad (1.0 \leqq CN \leqq 2.0)$$

$$Mo = \frac{N}{Ac} \cdot \frac{Ic}{y}$$

Ac : Area of pile
Ic : Moment of inertia of pile
y = Distance from center of pile to the edge of tensile sid

4.3 Verification for Shear Stress

When the mean shear stress in the concrete exceeds the allowable shear stress τa1, diagonal tension bars with a cross-sectional area more than calculated the equation below shall be placed.

$$Awreq = \frac{1.15 \cdot Sh' \cdot s}{\sigma sa \cdot d}$$

-Aw (cm^2) : Amount of diagonal tension bars at section where shear force worked
-s (cm) : Spacing of diagonal tension bars
-Awreq (cm^2) : Required amount of diagonal tension bars
- Sh' : Shear strength carried by diagonal tension bars
  Sh'＝S－Sca (kN)
- Sca : Shear strength carried by concrete
  Sca＝τa1・b・d (kN)

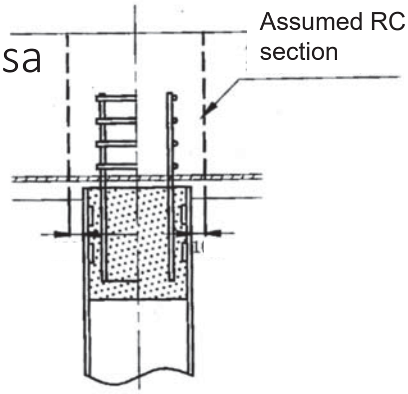
5. Connection between Pile and Footing

5.1 Verification of Assumed RC Section

Connection between pile and footing shall generally be rigid connections at pile heads, and the stress at connection shall be verified.

The stresses in the concrete and reinforcing bars in the footing is reviewed by assuming a virtual RC pile section in the footing.

Tensile stress $\sigma s \leqq$ Allowable stress σsa



Diameter of assumed RC section

Do = D＋ min of <0.25D+100> or <400> (mm)

= 2400.00 (mm)

5.2 Length of Pile Head Reinforcing Bar

Anchoring length bars into footing inside shall be more than L as indicated below, from the center of the lower main reinforcing bar in footing.

$$L \geqq Lo + 10 \cdot \phi$$

Where,

L : necessary length of bars in footing inside (mm)

Lo : necessary anchoring length of bars (mm)

$$Lo = \frac{\sigma sa}{4 \tau oa} \cdot \phi$$

σsa : allowable tensile stress of bars (N/mm^2)

τoa : allowable bond stress of concrete (N/mm^2)

φ : bar diameter(mm)

EXERCISE for DESIGN OF CAST-IN PLACE RC PILE

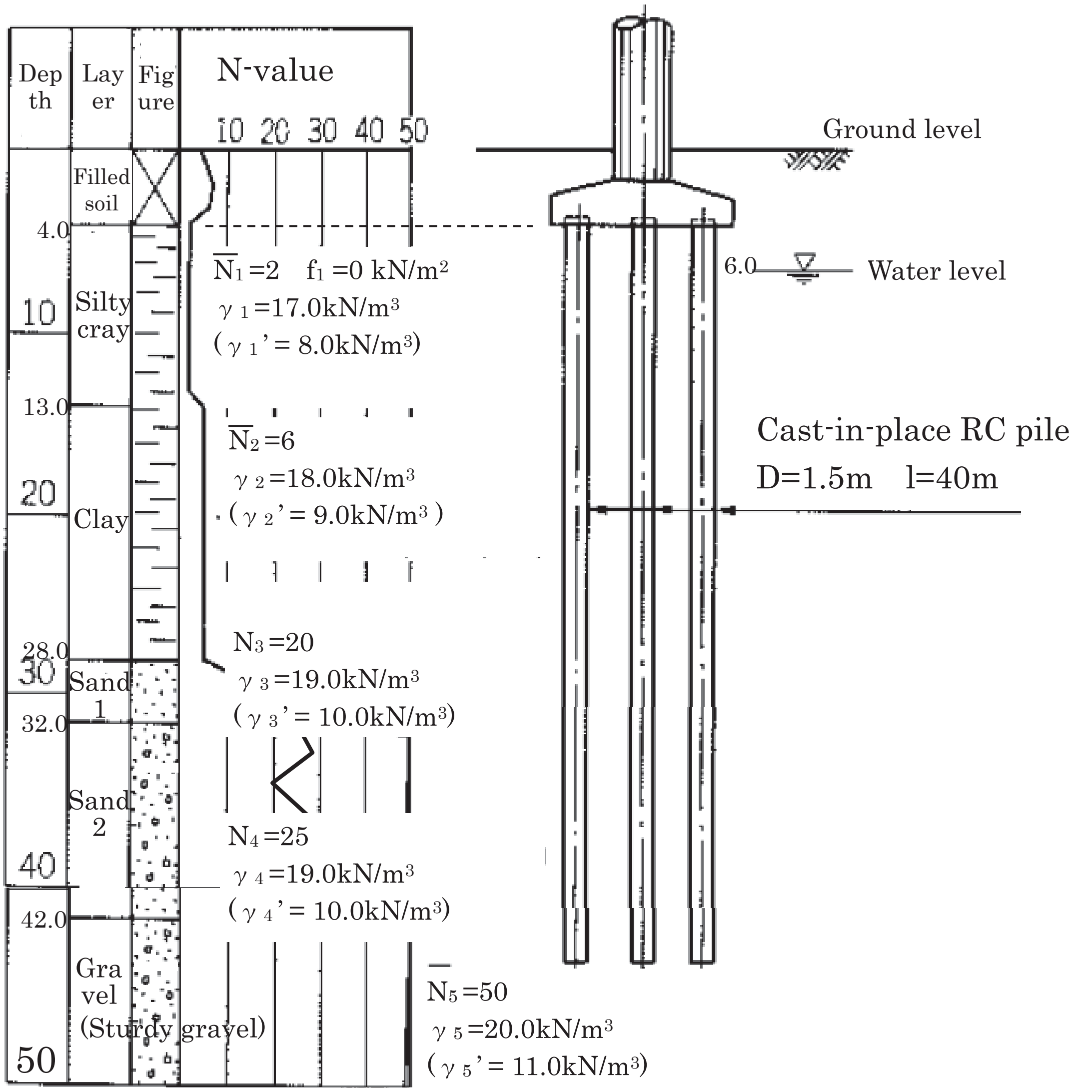
DESIGN CONDITION

(1) Outline of Foundation

Cast-in-place RC pile

- Pile diameter : $D = 1.5m$
- Pile length: $L = 40.0m$
- Unit weight of RC concrete : 24.5 kN/m^3

(2) Geological Condition



Note 1)

Shaft resistance intensity f_l for silty clay layer is evaluated as 0 kN/m^2.

Note 2)

Liquefaction are not estimated in the ground.

QUESTION. 1

Calculate following pile capacities by use of empirical bearing capacity estimation formula.

Each shall be obtained for Ordinary case and Seismic case, respectively.

- Allowable bearing capacity of pile
- Allowable pull-out capacity of pile

・End bearing capacity intensity (Cast-in-placed RC pile)

Ground Type	Ultimate Bearing Capacity End Bearing Intensity (kN/m²)
Gravelly Layer and Sandy Layer ($N \geq 30$)	3,000
Sturdy Gravelly Layer ($N \geq 50$)	5,000
Hard Cohesive Soil Layer	$3q_u$

Notes) q_u : unconfined compressive strength (kN/m²),
N : N value from the Standard Penetration Test (SPT)

・Maximum shaft resistance intensity

Pile Installation Method \ Ground Type	Sandy Soil	Cohesive Soil
Driven Pile Method (including Vibro-hammer Method)	$2N$ (≤ 100)	c or $10N$ (≤ 150)
Cast-in-place RC Pile Method	$5N$ (≤ 200)	c or $10N$ (≤ 150)
Bored Pile Method	$2N$ (≤ 100)	$0.8c$ or $8N$ (≤ 100)
Pre bored Pile Method	$5N$ (≤ 150)	c or $10N$ (≤ 100)
Steel Pipe Soil Cement Pile Method	$10N$ (≤ 200)	c or $10N$ (≤ 200)

(Note) c: cohesion of ground (kN/m²), N: N value from SPT.

ANSWER. 1

(1) Allowable bearing capacity

$$R_a = \frac{\gamma}{n}(R_u - W_s) + W_s - W$$

γ : 1.0

n : for ordinary case : 3, for seisimic case : 2

Ru = $q_d A + U \cdot \Sigma l_i f_i$

qd = 5,000 kN/m2

A = $\pi \times 0.75 \times 0.75$ =1.766 m2

U = $\pi \times D = \pi \times 1.5$ =4.71 m

i	Depth (m)	Thickness l_i (m)	Soil type	Averaged N-value	Shaft resistance intensity f_i (kN/m^2)	$l_i \cdot f_i$ (kN/m)	Unit weight γ_i (kN/m^3)	Effective weight γ_i' (kN/m^3)
1	4.0-6.0	2.0	Silty clay	2	0	0	17.0	17.0
2	6.0-13.0	7.0	Silty clay	2	0	0	17.0	8.0
3	13.0-28.0	15.0	Clay	6	60	900	18.0	9.0
4	28.0-32.0	4.0	Sand1	20	100	400	19.0	10.0
5	32.0-42.0	10.0	Sand2	25	125	1250	19.0	10.0
6	42.0-42.5	0.5	Gravel	50	200	100	20.0	11.0
6’	42.5-44.0	1.5	Gravel	50	200	(300)	20.0	11.0
Total		40.0	-	-	-	2,650 (2,950)	-	-

Note: ( ) is applied for allowable pull-out force.

Ru = 5,000 × 1.766 + 4.71 × 2,650 = 21,311.5 kN

Ws : $\pi \times 0.75 \times 0.75 \times (2.0 \times 17.0$ +7.0 × 8.0 +15.0 × 9.0 +4.0 × 10.0

+10.0×10.0 +0.5×11.0 +1.5×11.0) =683.5kN

W : π ×0.75×0.75×{38×(24.5－10.0) +2×24.5} = 1059.8kN

i) For ordinary case

Ra = 1/3×(21,311.5 – 683.5) + 683.5 – 1059.8 = 6,500 kN/pile

ii) For seismic case

Ra = 1/2×(21,311.5 – 683.5) + 683.5 – 1059.8 = 9,938 kN /pile

(2) Allowable pull-out force

$$P_a = \frac{1}{n} \cdot P_u + W$$

n : for ordinary case : 6, for seisimic case : 3

Pu =4.71×(2,950) = 13,894.5 kN

W : π ×0.75×0.75×{38×(24.5－10.0) +2×24.5} = 1059.8kN

i) For ordinary case

Pa = 1/6×13,894.5 + 1059.8 = 3,376 kN/pile

ii) For seismic case

Pa = 1/3×13,894.5 + 1059.8 = 5,691 kN/pile

QUESTION. 2

(1) Calculate a coefficient of horizontal subgrade reaction

(2) Determine the value of β. In addition, confirm the pile length within semi-infinite length of pile ($\beta \cdot L \geqq 3$)

(3) Calculate axial spring constant of pile

(4) Calculate radial spring constants of pile

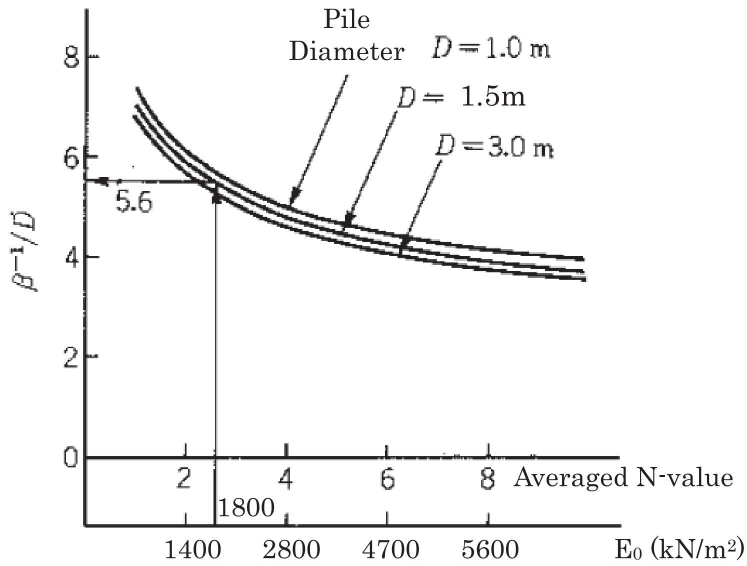
(5) Calculate stiffness matrix of pile for bridge longitudinal direction by use of displacement method

Each shall be calculated for both of Ordinary case and Seismic case.

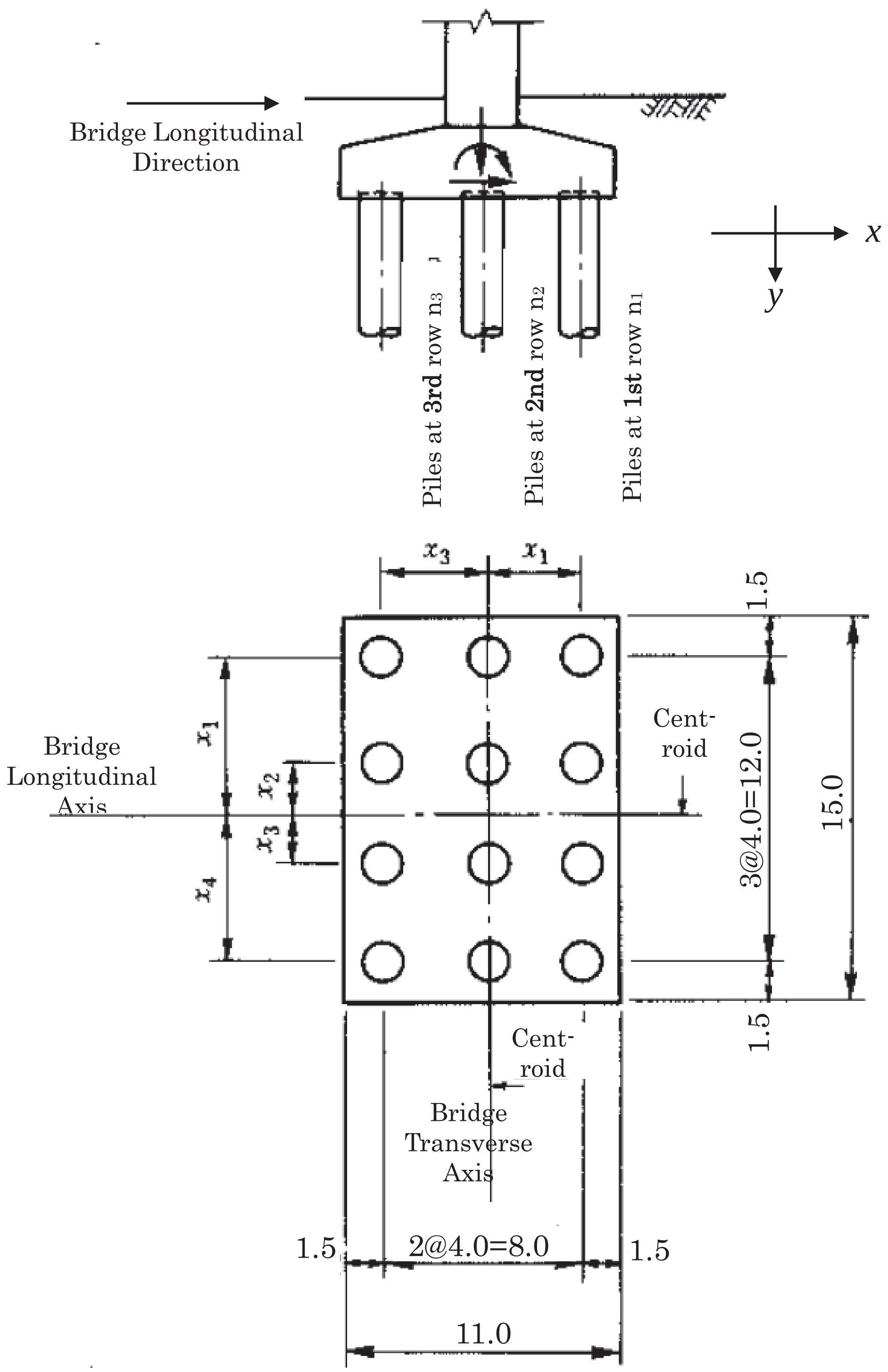
Supplemental Design Condition

・Modulus of deformation of ground (E_0) for silty clay layer was evaluated by unconfined compression test result ; E_0 = 1,800 kN /m^2

・Use the Figure below for tentative value of β^{-1}/ D in order to obtain the coefficient of horizontal subgrade reaction for ordinary case.



・Pile arrangement



・Displacement at the origin point can be obtained by following equations ;

$$A_{xx} \bullet \delta_x + A_{xy} \bullet \delta_y + A_{x\alpha} \bullet \alpha = H_0$$
$$A_{yx} \bullet \delta_x + A_{yy} \bullet \delta_y + A_{y\alpha} \bullet \alpha = V_0$$
$$A_{\alpha x} \bullet \delta_x + A_{\alpha y} \bullet \delta_y + A_{\alpha\alpha} \bullet \alpha = M_0$$

・Each coefficient can be obtained by following equations ;

$$A_{xx} = \sum\left(K_1 \bullet \cos^2\theta_i + K_V \bullet \sin^2\theta_i\right)$$
$$A_{xy} = A_{yx} = \sum\left(K_V - K_1\right) \bullet \sin\theta_i \bullet \cos\theta_i$$
$$A_{x\alpha} = A_{\alpha x} = \sum\left\{\left(K_V - K_1\right)x_i \bullet \sin\theta_i \bullet \cos\theta_i - K_2 \bullet \cos\theta_i\right\}$$
$$A_{yy} = \sum\left(K_V \bullet \cos^2\theta_i + K_1 \bullet \sin^2\theta_i\right)$$
$$A_{y\alpha} = A_{\alpha y} = \sum\left\{\left(K_V \bullet \cos^2\theta_i + K_1 \bullet \sin^2\theta_i\right)x_i + K_2 \bullet \sin\theta_i\right\}$$
$$A_{\alpha\alpha} = \sum\left\{\left(K_V \bullet \cos^2\theta_i + K_1 \bullet \sin^2\theta_i\right)x_i^2 + \left(K_2 + K_3\right)x_i \bullet \sin\theta_i + K_4\right\}$$

where

- H_0: lateral loads acting above a footing bottom (kN)
- V_0: vertical loads acting above a footing bottom (kN)
- M_0: moment of external forces around the origin O (kN·m)
- δ_x: lateral displacement at the origin O (m)
- δ_y: vertical displacement at the origin O (m)
- α: rotational angle of the footing (rad)
- x_i: x coordinate of the i-th pile head (m)
- θ_i: angle of a vertical axis from the i-th pile axis (degree). Signs to be in accordance with Fig.-C. 12.7.2.

ANSWER. 2

(1) Coefficient of horizontal subgrade reaction

$$k_H = k_{H0}\left(\frac{B_H}{0.3}\right)^{-\frac{3}{4}} \quad (\text{kN/m}^3)$$

$B_H = \sqrt{}(D/\beta) = \sqrt{}(\beta\text{-}1/D \cdot D2) = \sqrt{}(\beta\text{-}1/D) \cdot D = \sqrt{}5.6 \text{ x } 1.5 = 3.55\text{m}$

$k_{H0} = 1 / 0.3 \cdot \alpha \cdot E_0$

For ordinary ; $K_H = 1/0.3 \times 4 \times 1{,}800 \times (3.55/0.3)^{-3/4} ≒ 3{,}768 \text{ kN/m}^3$

For seismic ; $K_H = 2 \times 3{,}768 ≒ 7{,}536 \text{ kN/m}^3$

(2) Determination of β and confirmation of the pile length

$$\beta = \sqrt[4]{\frac{k_H D}{4EI}}$$

D = 1.5m

$E = 2.5 \text{ x } 10^7 \text{ (kN/m}^2\text{)}$

$I = \pi \cdot D^4 /64 = \pi /64 \times 1.5^4 = 0.2484 \text{ m}^4$

For ordinary ; $\beta = 0.12282 \text{ m}^{-1}$

For seismic ; $\beta = 0.14606 \text{ m}^{-1}$

$\beta \cdot L = 0.12282 \times 40 = 4.91 \geqq 3.0$

Therefore, the pile can be designed as semi-infinite length of pile.

(3) Axial spring constant of pile

$$K_V = a \cdot \frac{A_p \cdot E_p}{l}$$

$a = 0.031 \times L/D - 0.15$

$= 0.031 \times (40.0/1.5) - 0.15 = 0.677$

$Ap = 1/4 \times \pi \times D^2 = 1/4 \times \pi \times 1.5^2 = 1.766\text{m}^2$

$Kv = 0.677 \times (1.766 \times 2.5 \times 10^7) /40 = 747{,}239 \text{ kN/m}$

(4) Radial spring constant of pile

a) Ordinary Case

$K_1 = 4EI\beta^3 = 4\times2.5\times10^7\times0.2484\times0.12282^3 = 46{,}021$ kN/m

$K_2\ (=K_3) = 2EI\beta^2 = 2\times2.5\times10^7\times0.2484\times0.12282^2 = 187{,}353$ kN/rad

$K_4 = 2EI\beta = 2\times2.5\times10^7\times0.2484\times0.12282 = 1{,}525{,}424$ kN・m/rad

b) Seismic Case

$K_1 = 4EI\beta^3 = 4\times2.5\times10^7\times0.2484\times0.14606^3 = 77{,}401$ kN /m

$K_2\ (=K_3) = 2EI\beta^2 = 2\times2.5\times10^7\times0.2484\times0.14606^2 = 264{,}962$ kN /rad

$K_4 = 2EI\beta = 2\times2.5\times10^7\times0.2484\times0.14606 = 1{,}814{,}065$ kN・m/rad

(5) Stiffness matrix of pile (For Longitudinal direction)

a) Ordinary Case

$A_{xx} = \Sigma\,(K_1\cos^2\theta_i + K_v\sin^2\theta_i) = (46{,}021\times1.0+0)\times12 = 552{,}252$ kN /m

$A_{xy} = A_{yx} = \Sigma\,(K_v - K_1)\sin\theta_i\cdot\cos\theta_i = 0$ kN /m

$A_{xa} = A_{ax} = \Sigma\,\{(K_v - K_1)\,x_i\cdot\sin\theta_i\cdot\cos\theta_i - K_2\cos\theta_i\}$

$= (0-187{,}353)\times12 = -2{,}248{,}236$ kN

$A_{yy} = \Sigma\,(K_v\cos^2\theta_i + K_1\sin^2\theta_i) = (747{,}239\times1.0+0)\times12 = 8{,}966{,}868$ kN /m

$A_{ya} = A_{ay} = \Sigma\,\{(K_v\cos^2\theta_i + K_1\sin^2\theta_i)\,xi + K_2\sin\theta_i\}$

$= \{(747{,}239\times1.0+0)\times4.0\}\times4 + \{(747{,}239\times1.0+0)\times0.0\}\times4$

$+\{(747{,}239\times1.0+0)\times(-4.0)\}\times4 = 0$ kN

$A_{aa} = \Sigma\,\{(K_v\cos^2\theta_i + K_1\sin^2\theta_i)\,x_i^2 + (K_2+K_3)x_i\cdot\sin\theta_i + K_4\}$

$= \Sigma\,\{K_v\,x_i^2 + K_4\}$

$= \{(747{,}239\times4.0^2+1{,}525{,}424\}\times4 + \{(747{,}239\times0^2+1{,}525{,}424\}\times4$

$+\{(747{,}239\times(-4.0)^2+1{,}525{,}424)\times4 = 113{,}951{,}680$ kN・m/rad

b) Seismic Case

$A_{xx} = (77{,}401\times1.0+0)\times12 = 928{,}812$ kN /m

$A_{xy} = A_{yx} = 0$ kN /m

$A_{xa} = A_{ax} = (0-264{,}962)\times12 = -3{,}179{,}544$ kN

$A_{yy} = 747{,}239\times12 = 8{,}966{,}868$ kN /m

$A_{ya} = A_{ay} = 0$ kN

$A_{aa} = \{(747{,}239\times4.0^2+1{,}814{,}065\}\times4 + \{747{,}239\times0^2+1{,}814{,}065\}\times4$

$+\{(747{,}239\times(-4.0)^2+1{,}814{,}065)\times4$

$= 117{,}415{,}372$ kN・m/rad

QUESTION. 3

External force at the center of footing bottom is summarized in table below.

With the condition, calculate followings for bridge longitudinal direction;

(1) Displacement at pile head

(2) Axial force, radial force, moment acting on the pile head

(3) Verify the structural stability of the piles

Load case (Ordinary /Seismic)	Vertical force V (kN)	Horizontal force H (kN)	Bending moment M (kN.m)
Ordinary	43,605	1,030	4,738
Seismic	38,005	11,309	39,259

Note: Above are external forces when bridge longitudinal direction is considered.

Supplemental Explanation

With the condition of all piles arranged in vertical and with rigid connection to the footing, the equations for displacement and each forces acting on the pile head can be simplified by equations below;

$$\delta_x = \frac{H_0 \cdot A_{\alpha\alpha} - M_0 \cdot A_{x\alpha}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

$$\delta_y = \frac{V_0}{A_{yy}}$$

$$\alpha = \frac{-H_0 \cdot A_{\alpha x} + M_0 \cdot A_{xx}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

$$P_{Ni} = K_V (\delta_y + \alpha x_i)$$

$$P_{Hi} = K_1 \cdot \delta_x - K_2 \alpha$$

$$M_{ti} = -K_3 \cdot \delta_x + K_4 \alpha$$

ANSWER. 3

(1) Displacement at pile head

a) Ordinary Case

$$\delta_x = \frac{H_0 \cdot A_{\alpha\alpha} - M_0 \cdot A_{x\alpha}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

={1,030×113,951,680−4,738 (−2,248,236)}／{552,252×113,951,680 – (−2,248,236)･(−2,248,236)}

= 2.21 mm

$$\delta_y = \frac{V_0}{A_{yy}}$$

= 43,605／8,966,868 =4.86 mm

$$\alpha = \frac{-H_0 \cdot A_{\alpha x} + M_0 \cdot A_{xx}}{A_{xx} \cdot A_{\alpha\alpha} - A_{x\alpha} \cdot A_{\alpha x}}$$

={−1,030×(−2,248,236) +4,738×552,252}／{552,252×113,951,680 – (−2,248,236)･(−2,248,236)}

= 0.0000852 rad

b) Seismic Case

δ_x ={11,309 × 117,415,372 –39,259 (− 3,179,544)} ／ {928,812 × 117,415,372 – (−3,179,544)･(−3,179,544)}

= 14.68 mm

δ_y = 38,005／8,966,868 =4.24 mm

α ={ − 11,309 × (− 3,179,544) +39,259 × 928,812} ／ {928,812 × 117,415,372 – (−3,179,544) ・ (−3,179,544)}

= 0.0007319 rad

(2) Force and moment acting on the pile head

$P_{Ni} = K_V(\delta_y + \alpha x_i)$

$P_{Hi} = K_1 \cdot \delta_x - K_2 \alpha$

$M_{ti} = -K_3 \cdot \delta_x + K_4 \alpha$

a) Ordinary Case

・ 1st row (i =1)

P_{N1} = 747,239×(0.00486 + 0.0000852×4.000) = 3,886.2 kN /pile

P_{H1} = 46,021×0.00221 – 187,353×0.0000852 = 85.7 kN /pile

M_{t1} =－187,353×0.00221 + 1,525,424×0.0000852 =－284.1 kN・m /pile

・ 2nd row (i =2)

P_{N2} = 747,239×(0.00486 + 0.0000852×0) = 3,631.6 kN /pile

P_{H2} = 46,021×0.00221 – 187,353×0.0000852 = 85.7 kN /pile

M_{t2} =－187,353×0.00221 + 1,525,424×0.0000852 =－284.1 kN・m /pile

・ 3rd row (i =3)

P_{N3} = 747,239×(0.00486 - 0.0000852×4.000) =3,376.9 kN /pile

P_{H3} = 46,021×0.00221 – 187,353×0.0000852 = 85.7 kN /pile

M_{t3} =－187,353×0.00221 + 1,525,424×0.0000852 =－284.1 kN・m /pile

b) Seismic Case

・ 1st row (i =1)

P_{N1} = 747,239×(0.00424 + 0.0007319×4.000) = 5,355.9 kN /pile

P_{H1} = 77,401×0.01468 – 264,962×0.0007319 = 942.3 kN /pile

M_{t1} =－264,962×0.01468 + 1,814,065×0.0007319 =－2,561.9 kN・m /pile

・ 2nd row (i =2)

P_{N2} = 747,239×(0.00424 + 0.0007319×0) = 3,168.3 kN /pile

P_{H1} = 77,401×0.01468 – 264,962×0.0007319 = 942.3 kN /pile

M_{t1} =－264,962×0.01468 + 1,814,065×0.0007319 =－2,561.9 kN・m /pile

・ 3rd row (i =3)

P_{N3} = 747,239 × (0.00424 - 0.0007319 × 4.000) = 980.7 kN /pile
P_{H1} = 77,401 × 0.01468 – 264,962 × 0.0007319 = 942.3 kN /pile
M_{t1} = −264,962 × 0.01468 + 1,814,065 × 0.0007319 = −2,561.9 kN・m /pile

(3) Verification of structural stability

δ_{xmax} (Ordinary) = 2.21 mm < δ_a'=15.0 mm
δ_{xmax} (Seismic) = 14.68 mm < δ_a'=15.0 mm

P_{Nmax} (Ordinary) : 3,886.2kN < R_a = 6,500kN
P_{Nmin} (Ordinary) : 3,376.9kN > P_a = 0 kN

P_{Nmax} (Seismic) : 5,355.9kN < R_a' = 9,938kN
P_{Nmin} (Seismic) : 980.7kN > P_a' = -5,691 kN

QUESTION. 4

(1) With refer the calculation formula for a pile of semi-infinite length by Hayashi-Chang, express the following formulas in relation with depth x(m).

a) Horizontal displacement for pile head Rigid connection
b) Bending moment for pile head Rigid connection
c) Shear force for pile head Rigid connection
d) Bending moment for pile head Hinged connection

Above shall be calculated for Ordinary case and Seismic case in direction of the bridge longitudinal.

(2) Calculate Maximum bending moment at underground

i) Maximum bending moment at underground with Rigid pile head
ii) Maximum bending moment at underground with Hinged pile head

(3) Express the formula of above a) to d) by tables and graphs in relation of depth x(m). For the graphs, indicate the maximum forces and its depths.

ANSWER. 4

(1) Calculate sectional forces

A. For Ordinary Case

a) Horizontal displacement for pile head rigid

$$y=\frac{H}{2EI\beta^3}e^{-\beta x}\{(1+\beta h_0)\cos\beta x-\beta h_0\sin\beta x\}$$

= 85.7 / $(2\times 2.5\times 10^7\times 0.2484\times 0.12282^3)\times e^{-0.12282x}\times$ {(1+0.12282 × (－284.1) /85.7) × cos(0.12282*x*) – 0.12282 × (－284.1) /85.7 × sin (0.12282*x*)}

=0.00372 exp (－0.12282*x*)･{0.5929cos(0.12282*x*)+0.4071sin(0.12282*x*)}

b) Bending moment for pile head rigid

$$M=-\frac{H}{\beta}e^{-\beta x}\{\beta h_0\cos\beta x+(1+\beta h_0)\sin\beta x\}$$

=－85.7/0.12282 × exp (－0.12282*x*) × {0.12282 × (－284.1) /85.7 cos (0.12282*x*) + (1+0.12282 × (－284.1) / 85.7 sin (0.12282*x*))

=－697.8 exp (－0.12282*x*)･(－0.4071cos(0.12282*x*)
+ 0.5929 sin (0.12282*x*))

c) Shear force for pile head rigid

$$S=-He^{-\beta x}\{\cos\beta x-(1+2\beta h_0)\sin\beta x\}$$

=－85.7 × exp (－0.12282*x*) × {cos(0.12282*x*) – (1+2 × 0.12282 × (－284.1 /85.7) × sin (0.12282*x*)}

=－85.7 exp (－0.12282*x*)･{cos(0.12282*x*)– 0.1857 sin (0.12282*x*)}

d) Bending moment for pile head hinged

$$M=-\frac{H}{\beta}e^{-\beta x}\sin\beta x$$

=－85.7/0.12282 × exp (－0.12282*x*) × sin(0.12282*x*)

=－697.8 × exp (－0.12282*x*) × sin(0.12282*x*)

B. For Seismic Case

e) Horizontal displacement for pile head rigid

$$y=\frac{H}{2EI\beta^3}e^{-\beta x}\{(1+\beta h_0)\cos\beta x-\beta h_0\sin\beta x\}$$

= 942.3 / (2×2.5×10^7×0.2484×0.14606^3)×$e^{-0.14606x}$×{(1+0.14606×(−2561.9) /942.3)×cos(0.14606x) – 0.14606×(−2561.9) /942.3×sin (0.14606x)}

=0.02435 exp (−0.14606x)・{0.6029cos(0.14606x)+0.3971sin(0.14606x)}

f) Bending moment for pile head rigid

$$M=-\frac{H}{\beta}e^{-\beta x}\{\beta h_0\cos\beta x+(1+\beta h_0)\sin\beta x\}$$

=−942.3/0.14606×exp (−0.14606x)×{0.14606×(−2561.9) /942.3 cos (0.14606x) + (1+0.14606×(−2561.9) / 942.3 sin (0.14606x))

=−6451.5 exp (−0.14606x)・(−0.3971cos(0.14606x)
+ 0.6029 sin (0.14606x))

g) Shear force for pile head rigid

$$S=-He^{-\beta x}\{\cos\beta x-(1+2\beta h_0)\sin\beta x\}$$

=−942.3×exp (−0.14606x)×{cos(0.14606x) – (1+2×0.14606×(−2561.9) /942.3) × sin (0.14606x)}

=−942.3 exp (−0.14606x)・{cos(0.14606x)– 0.2057 sin (0.14606x)}

h) Bending moment for pile head hinged

$$M=-\frac{H}{\beta}e^{-\beta x}\sin\beta x$$

=−942.3/0.14606×exp (−0.14606x)×sin(0.14606x)

=−6451.5×exp (−0.14606x)×sin(0.14606x)

(2) Maximum bending moment at underground

A. For Ordinary Case

(i) Pile head Rigid

$$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h_0}$$

= 1/0.12282×tan^{-1}{1/(1+2×0.12282×(−284.1/85.7))}

=11.294m

$$M_m = -\frac{H}{2\beta}\sqrt{(1+2\beta h_0)^2+1}\cdot\exp(-\beta l_m)$$

=−85.7/(2×0.12282)×√{(1+2×0.12282×(−284.1)/ 85.7)^2+1}

×exp(−0.12282×11.294)

=−88.6 kN・m

(ii) Pile head Hinged

$$l_m = \frac{\pi}{4\beta}$$ = π / (4×0.12282) =6.391 m

$$M_m = -0.3224\frac{H}{\beta}$$ = −0.3224×85.7 / 0.12282 = - 225.0 kN・m

B. For Seismic Case

(iii) Pile head Rigid

$$l_m = \frac{1}{\beta}\tan^{-1}\frac{1}{1+2\beta h_0}$$

= 1/0.14606×tan^{-1}{1/(1+2×0.14606×(−2561.9/942.3))}

=9.365m

$$M_m = -\frac{H}{2\beta}\sqrt{(1+2\beta h_0)^2+1}\cdot\exp(-\beta l_m)$$

=−942.3/(2×0.14606)×√{(1+2×0.14606×(−2561.9)/ 942.3)^2

+1}×exp(−0.14606×9.365)

=−838.6 kN・m

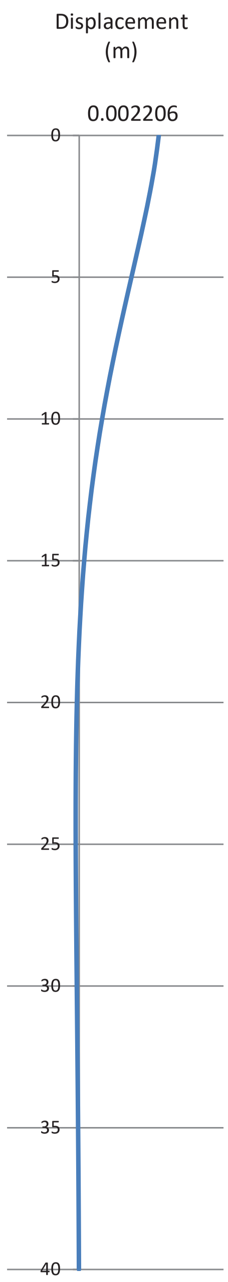
(iv) Pile head Hinged

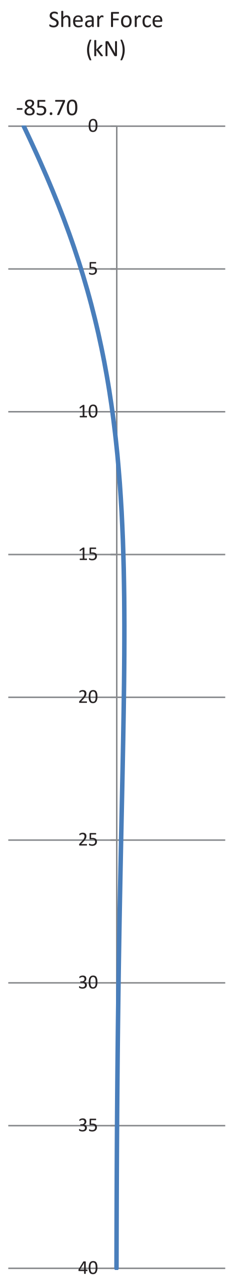
$$l_m = \frac{\pi}{4\beta}$$ = π / (4×0.14606) =5.375 m

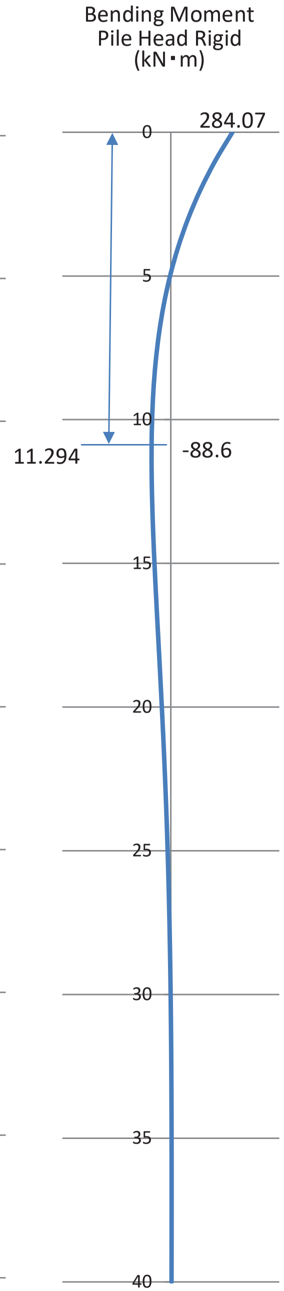
$$M_m = -0.3224\frac{H}{\beta}$$ = −0.3224×942.3 / 0.14606 = - 2080.0 kN・m

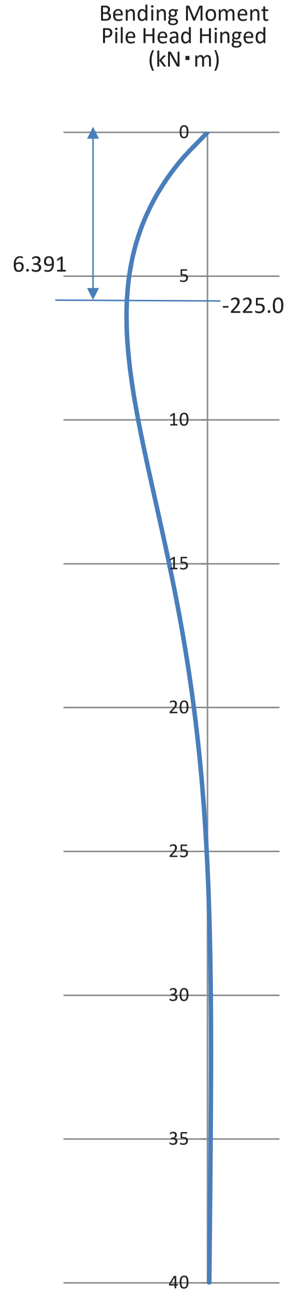
For Ordinary Case

	Pile Head Rigid			Pile Head Higed
Depth (m)	Horizontal displacement y (m)	Shear force (kN)	Bending Moment M (kN・m)	Bending Moment M (kN・m)
0	0.002206	-85.70	284.07	0.00
1	0.002100	-73.50	204.52	-75.61
2	0.001961	-62.00	136.84	-132.73
3	0.001801	-51.34	80.24	-173.87
4	0.001627	-41.64	33.83	-201.41
5	0.001448	-32.94	-3.37	-217.58
6	0.001269	-25.25	-32.38	-224.42
7	0.001095	-18.57	-54.21	-223.79
8	0.000930	-12.85	-69.84	-217.32
9	0.000776	-8.03	-80.20	-206.46
10	0.000635	-4.04	-86.16	-192.45
11	0.000507	-0.82	-88.53	-176.37
12	0.000394	1.73	-88.02	-159.08
13	0.000295	3.67	-85.27	-141.31
14	0.000210	5.09	-80.84	-123.64
15	0.000137	6.07	-75.23	-106.52
16	0.000077	6.67	-68.82	-90.28
17	0.000028	6.97	-61.98	-75.18
18	-0.000011	7.01	-54.97	-61.35
19	-0.000042	6.86	-48.02	-48.91
20	-0.000064	6.55	-41.31	-37.86
21	-0.000080	6.14	-34.95	-28.21
22	-0.000091	5.66	-29.05	-19.92
23	-0.000096	5.12	-23.66	-12.89
24	-0.000098	4.57	-18.81	-7.05
25	-0.000097	4.02	-14.51	-2.30
26	-0.000094	3.48	-10.76	1.48
27	-0.000088	2.96	-7.55	4.40
28	-0.000082	2.48	-4.83	6.56
29	-0.000075	2.04	-2.57	8.08
30	-0.000067	1.64	-0.74	9.05
31	-0.000059	1.28	0.71	9.57
32	-0.000052	0.96	1.83	9.72
33	-0.000044	0.69	2.66	9.58
34	-0.000037	0.46	3.23	9.21
35	-0.000031	0.27	3.60	8.68
36	-0.000025	0.11	3.78	8.03
37	-0.000020	-0.01	3.83	7.31
38	-0.000015	-0.11	3.76	6.55
39	-0.000011	-0.19	3.61	5.78
40	-0.000008	-0.24	3.40	5.03









For Seismic Case

	Pile Head Rigid			Pile Head Higed
Depth (m)	Horizontal displacement y (m)	Shear force (kN)	Bending Moment M (kN･m)	Bending Moment M (kN･m)
0	0.014681	-942.30	2561.89	0.00
1	0.013767	-781.20	1701.00	-811.36
2	0.012577	-632.11	995.48	-1387.27
3	0.011224	-497.48	431.98	-1766.15
4	0.009801	-378.61	-4.69	-1983.93
5	0.008376	-275.90	-330.58	-2073.39
6	0.007004	-189.03	-561.73	-2063.75
7	0.005721	-117.20	-713.61	-1980.47
8	0.004552	-59.25	-800.70	-1845.26
9	0.003512	-13.79	-836.21	-1676.27
10	0.002605	20.66	-831.90	-1488.29
11	0.001832	45.63	-798.01	-1293.06
12	0.001188	62.58	-743.28	-1099.61
13	0.000663	72.94	-675.01	-914.62
14	0.000246	77.98	-599.15	-742.77
15	-0.000074	78.87	-520.41	-587.07
16	-0.000310	76.62	-442.44	-449.17
17	-0.000475	72.12	-367.90	-329.64
18	-0.000581	66.10	-298.68	-228.24
19	-0.000638	59.17	-235.99	-144.11
20	-0.000658	51.82	-180.47	-75.98
21	-0.000648	44.41	-132.37	-22.30
22	-0.000617	37.25	-91.57	18.60
23	-0.000571	30.52	-57.73	48.45
24	-0.000515	24.38	-30.33	68.95
25	-0.000455	18.89	-8.76	81.72
26	-0.000393	14.09	7.67	88.24
27	-0.000332	10.00	19.66	89.86
28	-0.000275	6.57	27.89	87.75
29	-0.000222	3.76	33.00	82.94
30	-0.000174	1.53	35.60	76.29
31	-0.000132	-0.19	36.23	68.52
32	-0.000095	-1.47	35.37	60.18
33	-0.000065	-2.37	33.42	51.74
34	-0.000040	-2.96	30.73	43.53
35	-0.000019	-3.28	27.59	35.80
36	-0.000003	-3.41	24.23	28.70
37	0.000009	-3.37	20.82	22.34
38	0.000017	-3.23	17.51	16.77
39	0.000023	-3.00	14.40	12.00
40	0.000027	-2.71	11.54	7.99