CHAPTER 3

RESULTS OF GEOTECHNICAL INVESTIGATIONS

CHAPTER 3 RESULTS OF GEOTECHNICAL INVESTIGATIONS

3.1 General

A series of geotechnical investigations were carried out at the proposed site of the Suez Canal Bridge which lies between Sinai Peninsula and the Egyptian Western Desert. The main purpose of this investigation was to evaluate the soil data at the proposed site enabling the designer to recommend the foundation types, foundation level, allowable soil bearing capacity and the precautions to be taken during the ground engineering construction.

The following site surveys and laboratory tests were carried out;

1) Bore hole surveys to depth ranging from 25.0 m to 55.0 m.
2) Standard penetration tests
3) Ground water level
4) Laboratory soil tests
    - Specific gravity
    - Grain size distribution
    - Liquid limit, plastic limit, natural water content
    - Unconfined compression tests
    - Consolidation tests
    - Chemical analysis of ground water

3.2 Geotechnical Formation

The stratification of the soil formation at the site can be summarized as follows (refer to Fig. 3.1);

- The underlying stratum consists of light gray to grayish yellow, medium to fine grained poorly graded, slightly calcareous sand with some traces of silt and traces of cemented sand. This sand formation is repeatedly inter-bedded with relatively thin silty clay layers particularly at the depth ranging from 12.0 to 25.0. The thickness of this clay layer is normally in the range of 1.0 m to 4.0 m but exceptionally reached about 10.0 m thick. Sand strata continue below the encountered clay formation.

- At the location of deep bore holes, the overlying sand strata continue beneath where a silty clay layer exists at intermediate depths or lower depths. The thickness of these clay layers varies from 1.0 m to about 6.5 m. The clay layers can be described as light gray to light brown, laminated, expansive silty clay with traces of sand pockets, traces of iron oxides and occasional traces of gypsum crystals.

- Occasionally, thin layers of limestone and sandstone were encountered.

- Final ground water level was measured wherever encountered in bore holes at 0.10 to 3.40 m depth below the existing ground surface at the time of site investigation (July 1996).

3.3 Evaluation of Soil Parameters

3.3.1 Sandy Soil

The soil penetration resistance, as represented by the N values, can be used for the in-situ strength and compressibility of sandy soil strata. Figures 3.2 to 3.13 present the recorded and corrected N values.

Table 3.1 shows the recommended sandy soil parameters based on the field and laboratory testing results.

Table 3.1 Sandy Soil Parameters

Depth (m)	R_D (%)	Ø	E' (kg/cm^2)
0.0 to 6.0	65	32	180
6.0 to 10.0	80	36	260
> 10.0	95	41	390

where

Rd : Relative Density

Ø : Internal Friction Angle

E : Young's Ratio

3.3.2 Clayey Soil

From the field and laboratory tests results, the value of the untrained shear strength of the clay layers can be estimated as 2.5 kg/cm^2

The modules of deformation of the encountered clay layers can be evaluated approximately as 340 kg/cm^2

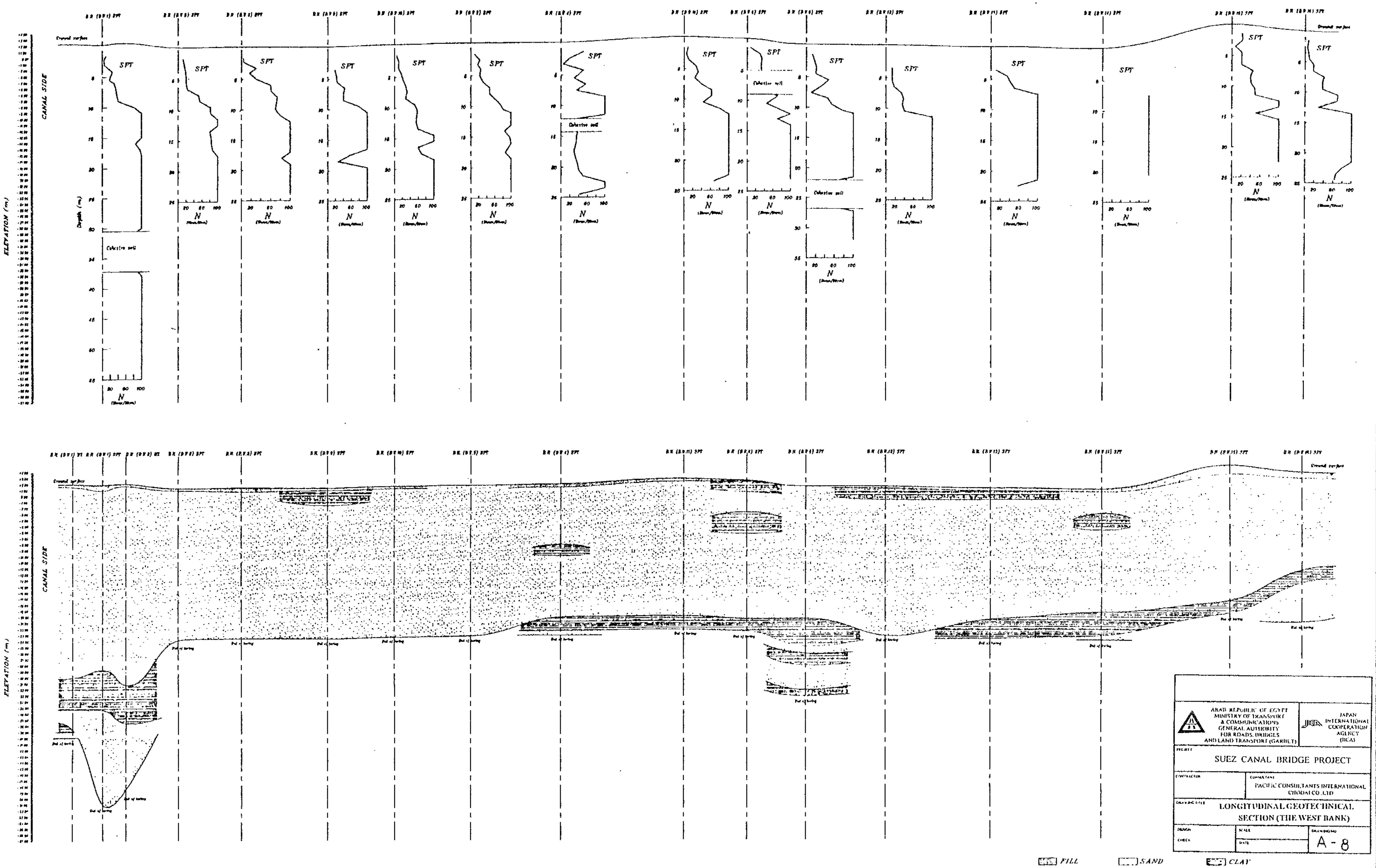
3.3.3 Chemical Analysis of Ground Water

Table 3.2 presents the chemical analysis of the ground water. The ground water near the Canal reflects the influence of the salt water of the Canal.

Table 3.2 Chemical Analysis of Ground Water

BH. NO.	T.D.S p.p.m.	SO3 p.p.m.	CL p.p.m.	PH
(B.W.1)	41045	2858	21286	6.89
(B.W.3)	4347	583	1974	7.47
(B.W.6)	803	105	367	7.77
(B.E.1)	31359	2923	15524	7.13
(B.E.6)	791	124	256	7.52

T.D.S : TOTAL DISSOLVED SALTS
SO3 : SULPHUR TRIOXIDE
CL : CHLORIDES
PH : HYDROGEN NUMBER



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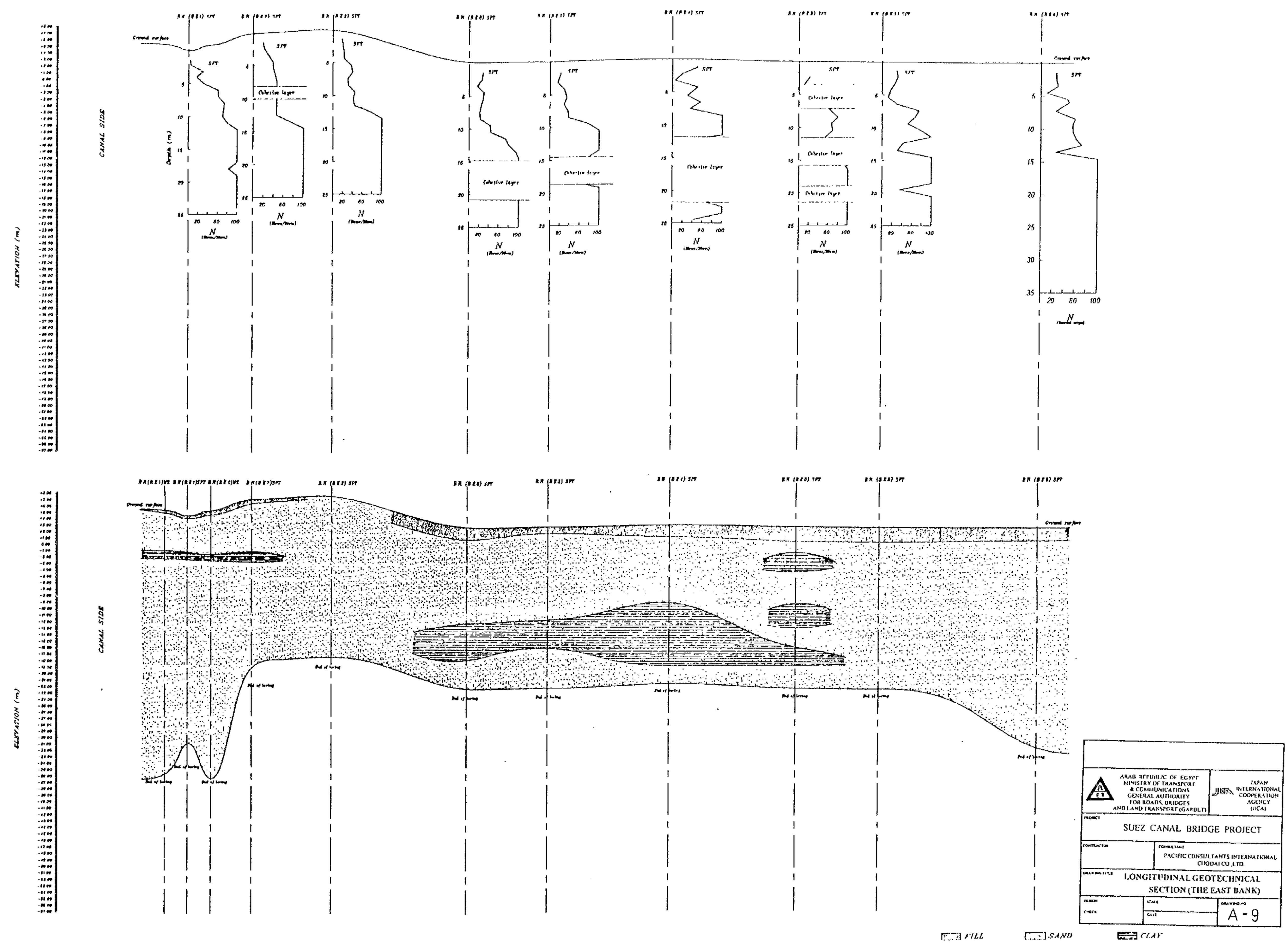
LONGITUDINAL GEOTECHNICAL SECTION
EL QANTRA BRIDGE OVER SUEZ CANAL
WEST SIDE

Fig. 3.1 (1) Simplified Longitudinal Geotechnical Section



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EAST SIDE

Fig. 3.1 (2) Simplified Longitudinal Geotechnical Section

CHAPTER 4

CALCULATION ANALYSIS FOR BRIDGES AND ROADS

CHAPTER 4 CALCULATION ANALYSIS FOR BRIDGES AND ROADS

4.1 Calculation Analysis for Main Bridge

4.1.1 Pylon Foundations

(1) Geological Conditions

Geological condition of each pylon is shown in Fig. 4.1.1 (at west bank) and 4.1.2 (at east bank).

Table 4.1.1 Soil Layers and Geotechnical Parameters

Side	Depth (m)		Soil Layer	Geotechnical Parameters				
	From	To		t/m^2	degree	E' t/m^2	C_u t/m^2	E_u t/m^2
East	G.S.	1.0	Fill					
	1.0	16.0	Top Sand	1.80	32	2000		
	16.0	36.0	Bottom Sand	1.85	36	3500		
West	G.S.	1.0	Fill					
	1.0	11.0	Top Sand	1.80	32	1500		
	11.0	30.5	Middle Sand	1.85	36	3500		
	30.5	38.5	Hard Clay	1.95			25.0	2500
	38.5	55.0	Bottom Sand	1.90	38	3500		

source : JICA Study Team Borehole and Soil Test Results

note : ground elevation ; East = 5.56 m

West = 2.08 m

The bottom level of the caissons (- 30 m) is decided to be not higher than -27.0 m (Suez Canal bottom elevation by expansion plan) and 3 meters penetration into bearing soil strata.

(2) Foundation Loads

Components of axial loads and moments in the three perpendicular axes X, Y and Z are given at the foundation level of + 0.5 m as follows;

PROJECT : Abridge Over Northern Part of Suez Canal
LOCATION : Kantara West Bank Km 48+517.5
OWNER : Jica Study Team

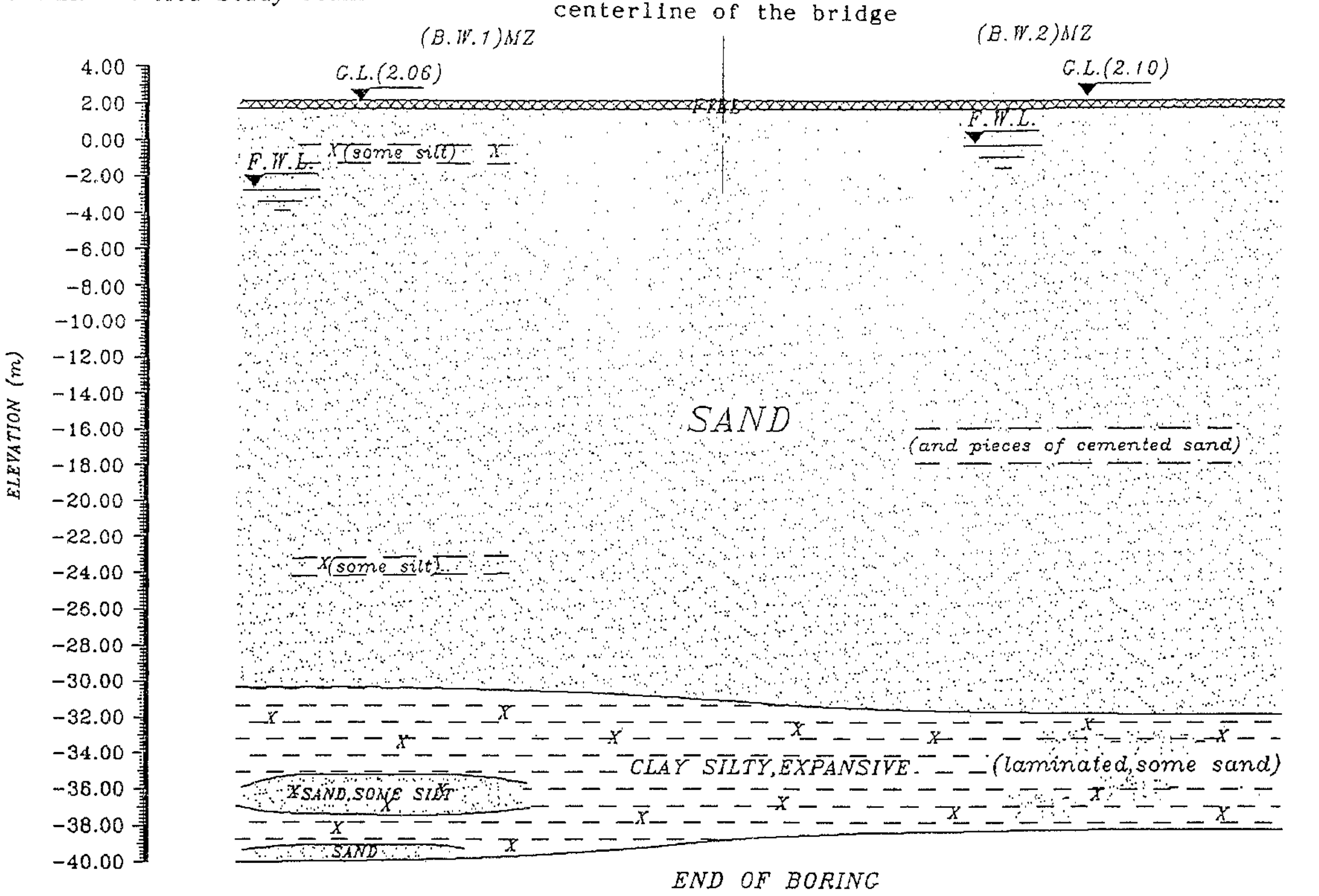


Fig. 4.1.1 Simplified Longitudinal Geotechnical Section

PROJECT : Abridge Over Northern Part of Suez Canal
LOCATION : Kantara East Bank Km 48+517.5
OWNER : Jica Study Team

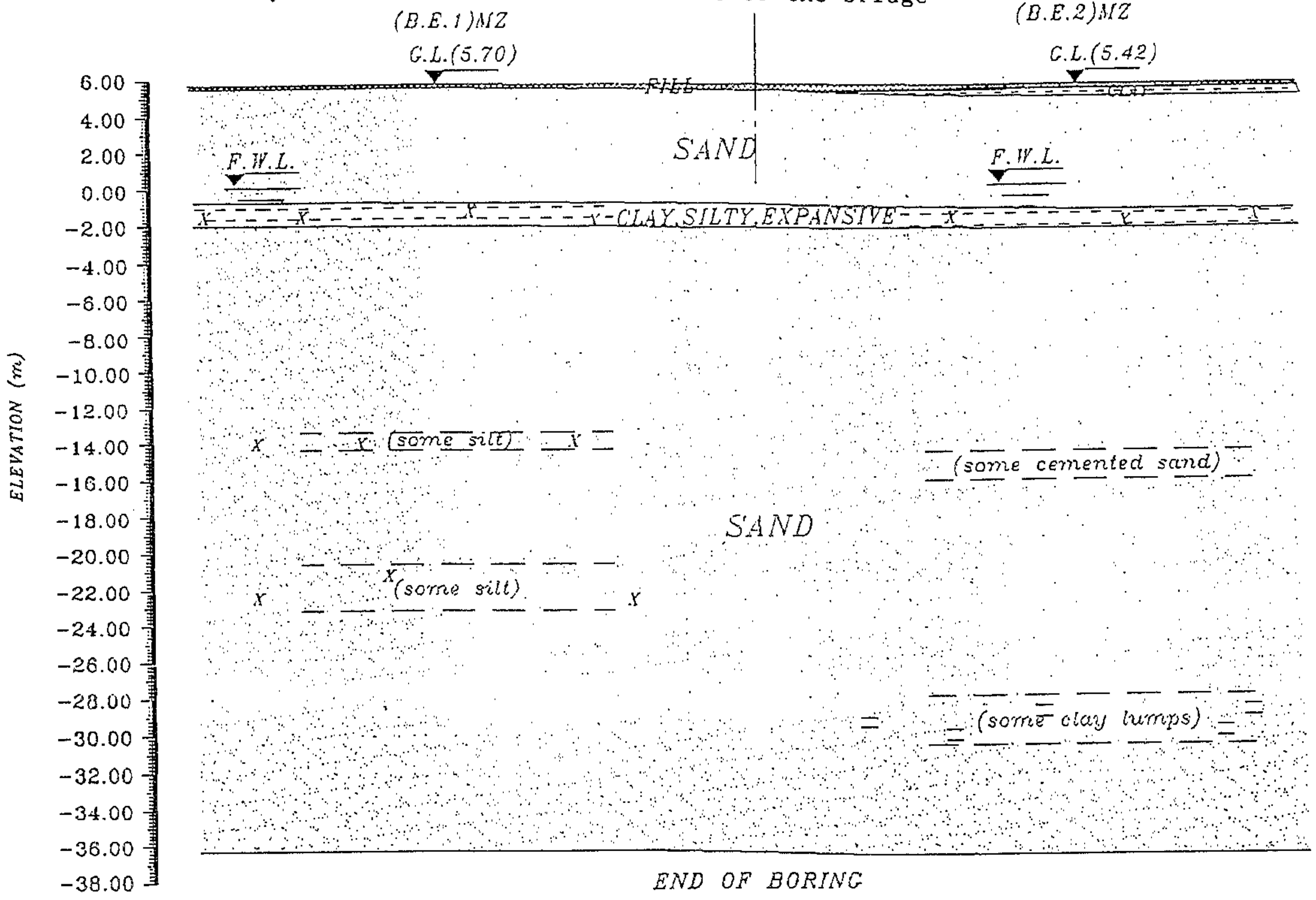


Fig. 4.1.2 Simplified Longitudinal Geotechnical Section

Table 4.1.2 Foundation Loads

Load Case	Axial Forces (ton)			Moments (tm)		
	Rx	RY	RZ	MX	MY	MZ
D+PS	7.9	12386.6	137.2	2446.6	-9.3	267.6
SD	3.9	37.4	0.4	6.7	9.2	588.6
L (RX max/min)	20.8	557.6	0.6	-117.7	-272.8	2040.3
	-16.9	727.3	-4.1	-159.8	450.8	1031.1
L (RY max/min)	4.3	1314.6	-4.4	-303.2	171.8	3153.5
	-0.6	-12.1	-0.6	48.7	20.1	-53.6
L (Mz max/min)	7.7	1173.7	-0.3	-130.2	298.2	3615.2
	-3.4	156.9	-5.4	-165.3	.-128.5	-573.8
W (X direction)	1.0	1329.2	525.5	22488.4	229.4	262.2
	-1.0	-1329.2	-501.4	-21866.3	-229.4	-262.2
W (Z direction)	255.1	21.1	0.5	10.8	-523.6	14837.8
EQ(X direction)	1.2	1842.5	1058.2	-34821.5	-257.7	-294.3
	-1.2	-1842.6	-1022.6	35739.5	257.7	294.0
EQ(Z direction)	1049.0	-17.6	0.9	25.0	-1948.1	52509.4
T	-50.9	1.5	-27.2	-654.2	134.1	-3602.4

(3) Stability of Foundations

The stability of the foundations is analyzed based on the Japanese Roadway Bridge Design Standard considering the interaction between the foundation and subsoil around/beneath it.

The allowable bearing capacity of soil for foundation embedded in sand and end on cohesive soil as the worst case of foundation on the west side is calculated using the following formula;

$$q_{all} = (3q_u + (\Sigma f_i L_i + \Sigma f_j' L_j')/t)/FS$$

where,

q_{all} = allowable bearing capacity of soil

q_u = unconfined compressive strength of cohesive soil layer at foundation tip

f_i = friction resistance in layer i
= 0.5 N <= 20 tm^2
L_i = thickness of soil layer i
f_j' = friction resistance to the inner surface of foundation in layer j
L_j' = thickness of soil layer j
= L_j = 0.3 L
L = embedded depth of foundation
t = thickness of diaphragm wall = 1.2 m
FS = safety factor
= 3 in normal case, 2 in earthquake or wind cases

1) Geotechnical data of West Bank :

Layer 1
Thickness = 7.5 m
SPT(N) = 40
Bulk unit weight = 1.80 t/m^3
Friction angle = 32 deg.
K_a (normal loading) = 0.3072588
K_p (normal loading) = 3.254585
K_p (earthquake) = 3.045692
K_p (normal loading) canal side = 2.717124
K_p (earthquake) canal side = 2.088943

Layer 2
Thickness = 5.0 m
SPT(N) = 40
Bulk unit weight = 1.80 t/m^3
Friction angle = 32 deg.
K_a (normal loading) = 0.3072588
K_p (normal loading) = 3.254585
K_p (earthquake) = 3.045692
K_p (normal loading) canal side = 2.717124
K_p (earthquake) canal side = 2.088943

Layer 3
Thickness = 18.0 m
SPT(N) = 100

Bulk unit weight = 1.85 t/m^3
Friction angle = 36 deg.
K_a (normal loading) = 0.2596165
K_p (normal loading) = 3.851835
K_p (earthquake) = 3.62885
K_p (normal loading) canal side = 3.805425
K_p (earthquake) canal side = 2.424265

Layer 4 (bearing soil layer)
qu = 60.8 t/m^2

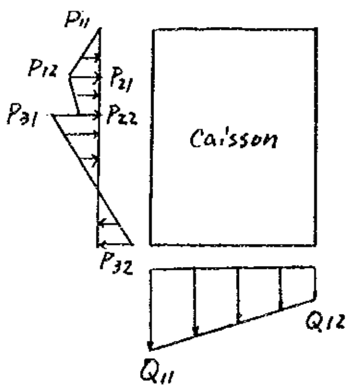
Allowable bearing capacity of soil at foundation tip ;
normal loading condition: 270 t/m^2
wind or earthquake loading condition: 405 t/m^2

2) Results of Stability Analysis

Canal side earthquake loading condition :

RX	=	1061.9 t
RY	=	12396.4 t
RZ	=	137.4 t
MX	=	2480.2 tm
MY	=	-1951.8 tm
MZ	=	53736.7 tm

PP11	=	0.0 t/m^2	FS =	> 1.1
PP12	=	10.7 t/m^2	FS =	1.17 > 1.1
PP21	=	10.7 t/m^2	FS =	1.17 > 1.1
PP22	=	7.4 t/m^2	FS =	2.83 > 1.1
PP31	=	22.0 t/m^2	FS =	1.10 > 1.1
PP32	=	-12.7 t/m^2	FS =	4.83 > 1.1
Q11	=	298.4 t/m^2	<	405 t/m^2
Q12	=	139.3 t/m^2	<	405 t/m^2



3) Settlement of the Caissons

Pylon foundation in the east bank shall be embedded and end in the thick sand layer down to the investigated depth of 42 m. Pylon foundation on the west bank shall be embedded in sand and ende on hard cohesive soil layer. Their settlements are estimated according to the Egyptian Code of Practice for Foundation Design and Construction, Vol. 2, Part 4 : Deep Foundation issued by the Ministry of Housing and Reconstruction, as shown if Fig. 4.1.3 for the foundation on the west bank.

Settlement of foundation on the east bank where soil is sand is estimated as below ;

$$S = q\,B\,(1-m^2)\quad I / E$$

where,

S = foundation settlement

q = intensity of stress at depth of 2/3 of the foundation depth

B = least lateral dimension of foundation : 12 m

m = Poisson's ratio : 0.3

I = influence factor : 0.82

E = elastic modulus : 14,000 t/m^2

Substituting in the above equation, the estimated settlement of foundation on the east bank is 5.5 cm, which shall be immediate during construction.

Settlement of foundation on the west bank is estimated as below;

Sand layer;

$$Ss_i = ds_i\,L_i / E_i$$

where,

Ss_i = settlement of layer i

ds_i = stress increment at middle of sand layer i

L_i = thickness of sand layer i

E_i = elastic modulus of sand layer i

Cohesive soil layer;

$$Sc_i = \{Cr_i / (1 + eo_i)\}\,L_i\,\mathrm{Log}\,\{(s_i + ds_i) / s_i\}$$

$$Sc_i = \{Cc_i / (1 + eo_i)\}\,L_i\,\mathrm{Log}\,\{(s_i + ds_i) / s_i\}$$

where,

Sc_i = settlement of layer i

Cr_i = recompression index of layer i

Cc_i = compression index of layer i

eo_i = initial void ratio of layer i

L_i = thickness of layer i

s_i = initial stress at middle of layer i

ds_i = stress increment at middle of layer i

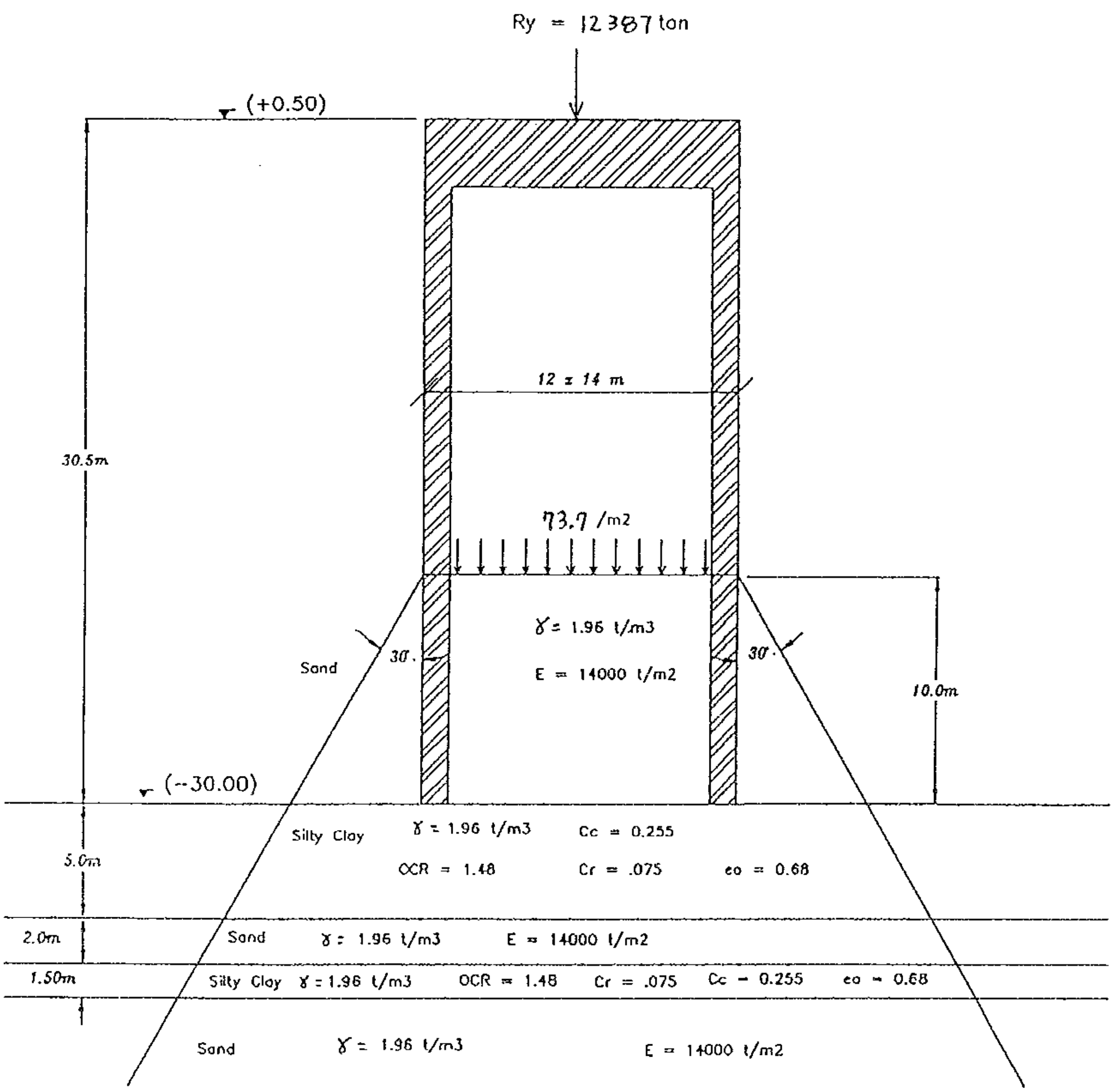


Fig. 4.1.3 Geometrical and Geotechnical Data for Settlement Estimation

Substituting in the above equations with dimensions, stress and soil parameters as shown in Fig. 4.1.3, settlement of foundation on the west bank is estimated as follows;

Table 4.1.3 Foundation Settlement on the West Bank

Parameter	Layer No.					
	1 (Sand)	2a (Cohes.)	2b Cohes.)	3 (Sand)	4 (Cohes.)	5 (Sand)
Cr_i		0.075	0.075		0.075	
Cc_i		0.255	0.255		0.255	
eo_i		0.68	0.68		0.68	
s_i (t/m^2)		33.9	36.6		40.1	
ds_i (t/m^2)	38.4	13.4	9.7	14.7	6.0	10.7
L_i (m)	10.0	3.0	2.0	2.0	1.5	5.0
E_i (t/m^2)	14000			14000		14000
Sand S_i (cm)	2.74			0.21		0.37
Clay S_i, recomp.(cm)		1.94	0.91		0.41	
Clay S_i, comp.(cm)		0.00	0.00		0.00	
Settlement of layer	2.74	1.94	0.91	0.21	0.41	0.37
Settlement of sand layer	3.32					
Settlement of clay layer	3.26					
Total settlement	6.58					

note: preconsolidation pressure ; layer 2 = 4.8 kg/cm^2, layer 4 = 6.0 kg/cm^2

The settlement of sand layer shall be immediate while settlement of cohesive layers shall be delayed.

4.1.2 Main Pylon

Stress conditions of earthquake cases are summarized in Table 4.1.4 and 4.1.5.

Table 4.1.4 Stress Condition (Longitudinal Direction)

Sect . No.	Acting Forces		Re-bar	Stress Results		Sectional Dimension (m)		
	N (ton)	M (tm)	(Dia x No.)	fc (kg/cm^2)	fs (kg/cm^2)	width	depth	wall thickness
4035	11095	54875	40 x 204	112	2561	7.0	8.0	-
4032	9595	48028	40 x 192	145	1866	6.582	7.862	0.7
4027	7015	4782	28 x 76	81	-	3.0	6.682	0.7
4025	6193	2685	25 x 72	86	-	3.0	6.682	0.7

Table 4.1.5 Stress Condition (Transverse Direction)

Sect . No.	Acting Forces		Re-bar	Stress Results		Sectional Dimension (m)		
	N (ton)	M (tm)	(Dia x No.)	fc (kg/cm^2)	fs (kg/cm^2)	width	depth	wall thickness
4035	9240	39597	32 x 228	105	2811	8.0	7.0	-
4032	7722	30457	32 x 220	126	2000	7.862	6.582	0.7
4027	8915	10327	32 x 184	150	589	6.682	3.0	0.7
4025	6705	4447	25 x 180	119	-	6.682	3.0	0.7

4.1.3 Pier Foundations

(1) Geological Conditions

Same geological condition is expected for the auxiliary piers. The bearing soil layer of fine cemented sand is set at elevation of -17.5 m.

Table 4.1.6 Geological Conditions for Pile Foundation

No.	Thickness (m)	EO (kg/cm^2) 28*N	Normal Case		Earthquake Case	
			KH (t/m^3)	beta (1/m)	KH (t/m^3)	beta (1/m)
1	2.2	84	555.9	0.13534	1111.7	0.16095
2	1.2	924	6114.6	0.24648	12229.1	0.29311
3	11.6	1400	9264.5	0.27346	18529.0	0.32520

(2) Acting Forces from Pile Cap

1) PM1/PM6

Acting Forces at pile group center

Case	Direction	Load Condition	V (t)	H (t)	M (tm)
1	long.	wind	7097.2	-169.4	-5708.6
2	long.	wind	7139.6	169.4	5708.6
3	long.	earthquake	7066.4	779.1	25231.4
4	long.	earthquake	7170.4	-665.1	-17891.3
5	transv.	wind	7068.2	-199.8	-9613.8
6	transv.	wind	7168.6	199.8	9613.8
7	transv.	earthquake	7118.4	764.9	24428.8

2) PM2 - PM5

Acting Forces at pile group center

Case	Direction	Load Condition	V (t)	H (t)	M (tm)
1	long.	wind	4385.2	-148.2	-5216.1
2	long.	earthquake	4491.4	568.2	18072.1
3	long.	earthquake	4279.0	-568.2	-18072.1
4	transv.	wind	4327.2	-175.2	-8590.5
5	transv.	wind	4443.2	175.2	8590.5
6	transv.	earthquake	4385.3	-501.0	-13816.3
7					

(3) Stability of Foundation

- Allowable bearing capacity of pile :
  normal case : 495 t/pile (280 t/m^2 at pile tip)
  earthquake : 743 t/pile

- pile arrangement
  diameter 1.5 m
  length 15.0 m
  number
  PM1/PM6 4×5 rows (transverse direction 5 piles)
  PM2-PM5 4×4 rows
  distance between piles : 3.8 m (> 2.5 × 1.5 m)

1) PM1/PM6

Stability Analysis Results

Case	Direction	Loading C.	Deformation. cm	Allowable Deform. cm	Pile Force. ton	Allowable Force. ton	Pulling Force. ton	Allowable Force. ton
1	Long.	Wind	- 0.13	1.50	441.6	495	0	- 440
2	Long.	Wind	0.13	1.50	443.8	495	0	- 440
3	Long	Earthq.	0.47	1.50	728.6	743	- 22.0	- 440
4	Long	Earthq.	- 0.37	1.50	628.7	743	0	- 440
5	Transv.	Wind	- 0.14	1.50	476.2	495	0	- 440
6	Transv.	Wind	0.14	1.50	481.2	495	0	- 440
7	Transv.	Earthq.	0.33	1.50	670.8	743	0	- 440

2) PM2-PM5

Stability Analysis Results

Case	Direction	Loading C.	Deformation. cm	Allowable Deform. cm	Pile Force. ton	Allowable Force. ton	Pulling Force. ton	Allowable Force. ton
1	Long.	Wind	- 0.24	1.50	412.1	495	0	
2	Long.	Earthq.	0.69	1.50	702.9	743	- 104.0	- 440
3	Long.	Earthq.	- 0.69	1.50	688.7	743	- 118.2	- 440
4	Transv.	Wind	- 0.17	1.50	436.3	495	0	
5	Transv.	Wind	0.17	1.50	444.0	495	0	
6	Transv.	Earthq.	- 0.24	1.50	535.3	743	0	- 440
7								

4.1.4 Piers

(1) Pier Columns

1) Acting Forces (PM1/PM6)

Summary of Acting Forces at Pier Column Base

Case	Direction	Loading Condition	Bending M. M (tm)	Axial F. N (t)	Shear F. S (t)
1	Long.	Wind	2600.17	2607.88	84.71
2	Long.	Wind	2600.17	2629.08	84.71
3	Long.	Earthquake	11595.23	2592.48	295.47
4	Long.	Earthquake	11595.23	2644.48	295.47
5	Long.	Wind	5223.88	2593.38	120.40
6	Transv.	Wind	3790.57	2643.58	79.40
7	Transv.	Earthquake	11098.00	2628.28	287.37
8	Transv.	Earthquake	11220.77	2608.68	289.37

2) Stress Condition (PM1/PM6)

Summary of Stress of Column Base Section

Case	Direction	Loading Condition	Concrete Stress		Re-bar Stress		Shear Stress	
			Acting	Allow.	Acting	Allow.	Acting	Allow.
1	Long.	Wind	17.4	87.5	9.7	2250	0.3	6.4
2	Long.	Wind	17.5	87.5	8.5	2250	0.3	6.4
3	Long.	Earthquake	79.1	105	2752	3000	0.8	5.9
4	Long.	Earthquake	79.1	105	2713	3000	0.8	5.9
5	Transv.	Wind	28.4	87.5	276	2250	0.4	5.6
6	Transv.	Wind	20.7	87.5	60	2250	0.2	6.1
7	Transv.	Earthquake	73.9	105	2677	3000	0.6	6.0
8	Transv.	Earthquake	74.9	105	2761	3000	0.7	6.0

3) Acting Forces (PM2-PM5)

Summary of Acting Forces at Pier Column Base

Case	Direction	Loading Condition	Bending M. M (tm)	Axial F. N (t)	Shear F. S (t)
1	Long.	Wind	2358.84	1502.72	74.08
2	Long.	Earthquake	8291.65	1555.82	215.13
3	Long.	Earthquake	8291.65	1449.62	215.13
4	Transv.	Wind	4917.10	1473.72	109.67
5	Transv.	Wind	3147.79	1531.72	65.54
6	Transv.	Earthquake	5779.46	1475.32	176.13
7	Transv.	Earthquake	6475.14	1530.22	186.93

4) Stress Condition (PM2-PM5)

Summary of Stress of Column Base Section

Case	Direction	Loading Condition	Concrete Stress		Re-bar Stress		Shear Stress	
			Acting	Allow.	Acting	Allow.	Acting	Allow.
1	Long.	Wind	20.2	87.5	169	2250	0.4	5.6
2	Long.	Earthquake	72.5	105	2477	3000	1.0	5.8
3	Long.	Earthquake	72.3	105	2552	3000	1.0	5.8
4	Transv.	Wind	48.8	87.5	1535	2250	0.5	5.1
5	Transv.	Wind	28.0	87.5	424	2250	0.3	5.4
6	Transv.	Earthquake	58.5	105	2123	3000	0.8	6.0
7	Transv.	Earthquake	66.0	105	2538	3000	0.9	5.9

(2) Pile Caps

1) PM1/PM6

Summary of Acting Force on Pile Cap

Case	Direction	Loading Condition	Bending Moment (tm)
1	Long.	Wind	- 92.66, 2881.32
2	Long.	Wind	- 92.66, 2898.28
3	Long.	Earthquake	- 995.85, 5292.24
4	Long.	Earthquake	- 175.39, 4489.86
5	Long.	Dead Load	- 66.10, 2169.00
6	Transv.	Wind	- 2019.63, 1537.09
7	Transv.	Wind	- 2500.56, 1509.98
8	Transv.	Earthquake	- 6249.47, 3858.47
9	Transv.	Earthquake	- 2762.69, 3810.16

Summary of Stress Condition of Pile Cap

Case	Direction	Loading Condition	Position	Concrete Stress		Re-bar Stress	
				Acting	Allow.	Acting	Allow.
1	Long.	Wind	Upper S.	1.3	87.5	111	2250
			Lower S.	31.6	87.5	1529	2250
2	Long.	Wind	Upper S.	1.3	87.5	111	2250
			Lower S.	31.8	87.5	1538	2250
3	Long.	Earthquake	Upper S.	14.2	105	1192	3000
			Lower S.	58.0	105	2808	3000
4	Long.	Earthquake	Upper S.	2.5	105	210	3000
			Lower S.	49.2	105	2383	3000
5	Long.	Dead Load	Upper S.	-	70	-	1400
			Lower S.	23.8	70	1151	1400
6	Transv.	Wind	Upper S.	22.9	87.5	967	2250
			Lower S.	17.4	87.5	1112	2250
7	Transv.	Wind	Upper S.	28.4	87.5	1197	2250
			Lower S.	17.1	87.5	1093	2250
8	Transv.	Earthquake	Upper S.	71.0	105	2992	3000
			Lower S.	43.7	105	2792	3000
9	Transv.	Earthquake	Upper S.	31.4	105	1323	3000
			Lower S.	43.2	105	2757	3000

2) PM2-PM5

Summary of Acting Force on Pile Cap

Case	Direction	Loading Condition	Bending Moment (tm)
1	Long.	Wind	- 177.12, 1425.06
2	Long.	Earthquake	- 1116.22, 2871.24
3	Long.	Earthquake	- 1224.03, 2869.31
4	Long.	Wind	- 118.48, 2833.55
5	Transv.	Wind	- 521.65, 300.39
6	Transv.	Earthquake	- 1021.53, 2430.62
7	Transv.	Earthquake	- 2793.11, 39.80

Summary of Stress Condition of Pile Cap

Case	Direction	Loading Condition	Position	Concrete Stress		Re-bar Stress	
				Acting	Allow.	Acting	Allow.
1	Long.	Wind	Upper S.	3.6	87.5	407	2250
			Lower S.	19.0	87.5	1198	2250
2	Long.	Earthquake	Upper S.	22.6	105	2563	3000
			Lower S.	38.4	105	2413	3000
3	Long.	Earthquake	Upper S.	24.8	105	2811	3000
			Lower S.	38.3	105	2412	3000
4	Transv.	Wind	Upper S.	1.9	87.5	126	2250
			Lower.S	39.7	87.5	2152	2250
5	Transv.	Wind	Upper S.	8.3	87.5	555	2250
			Lower S.	4.2	87.5	228	2250
6	Transv.	Earthquake	Upper S.	16.2	105	1086	3000
			Lower S.	34.1	105	1846	3000
7	Transv.	Earthquake	Upper S.	44.3	87.5	2967	3000
			Lower S.	0.6	87.5	2226	3000

4.1.5 Steel Deck

(1) Structure Skeleton for Sectional Force Analysis

Figure 4.1.4 to 4.1.6 shows the 3 - dimensional skeleton of the main bridge for stress analysis. The main deck girder is represented single beam having the lateral ribs of which both ends are supported by stay cables from the main pylons.

The foundations of auxiliary piers and main pylons are represented by composite springs of 6 directions (Dx, Dy, Dz, Tx, Ty and Tz).

The support condition at pylons and auxiliary piers are as below ;

main pylon :	vertical	- fix,
	longitudinal	- spring,
	transverse	- fix,
	moment (X)	- free,
Pier :	vertical	- fix,
	longitudinal	- free,
	transverse	- fix, and
	moment (X)	- free.

(2) Sectional Forces by Loading

Fig. 4.1.7 to 4.1.11 shows the envelope of maximum and minimum sectional forces along the girder axis under the combination of the loadings.

(3) Stress Condition of Major Girder Sections

Fig. 4.1.12 shows of steel girder formation. Table 4.1.7 to 4.1.9 summarizes the girder sectional stress level of major girder section under most unfavorite loading combination.

Main Bridge Skelton (1) S = 1/1000

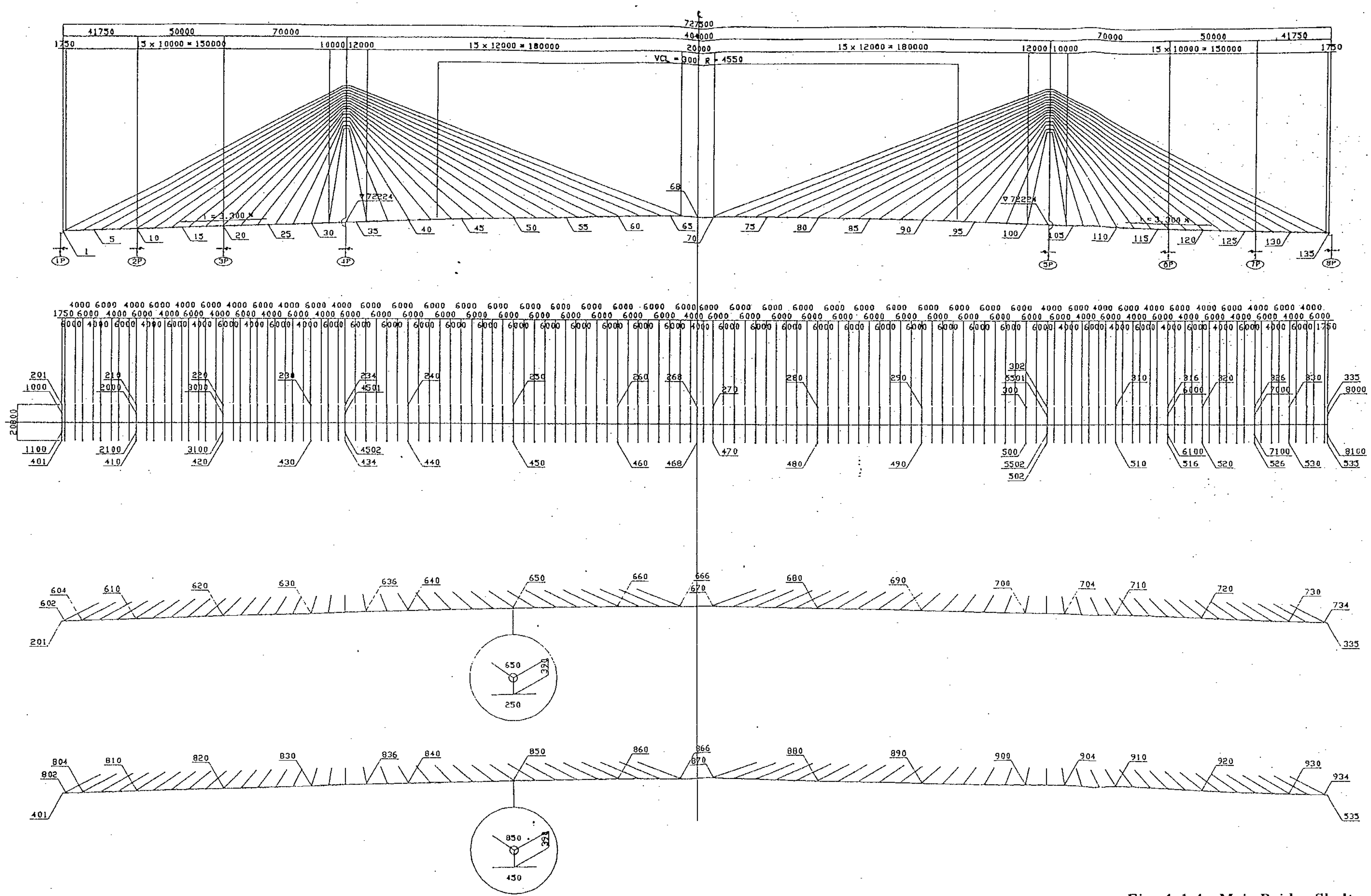


Fig. 4.1.4 Main Bridge Skelton (1)

Main Bridge Skelton (2) S = 1/500

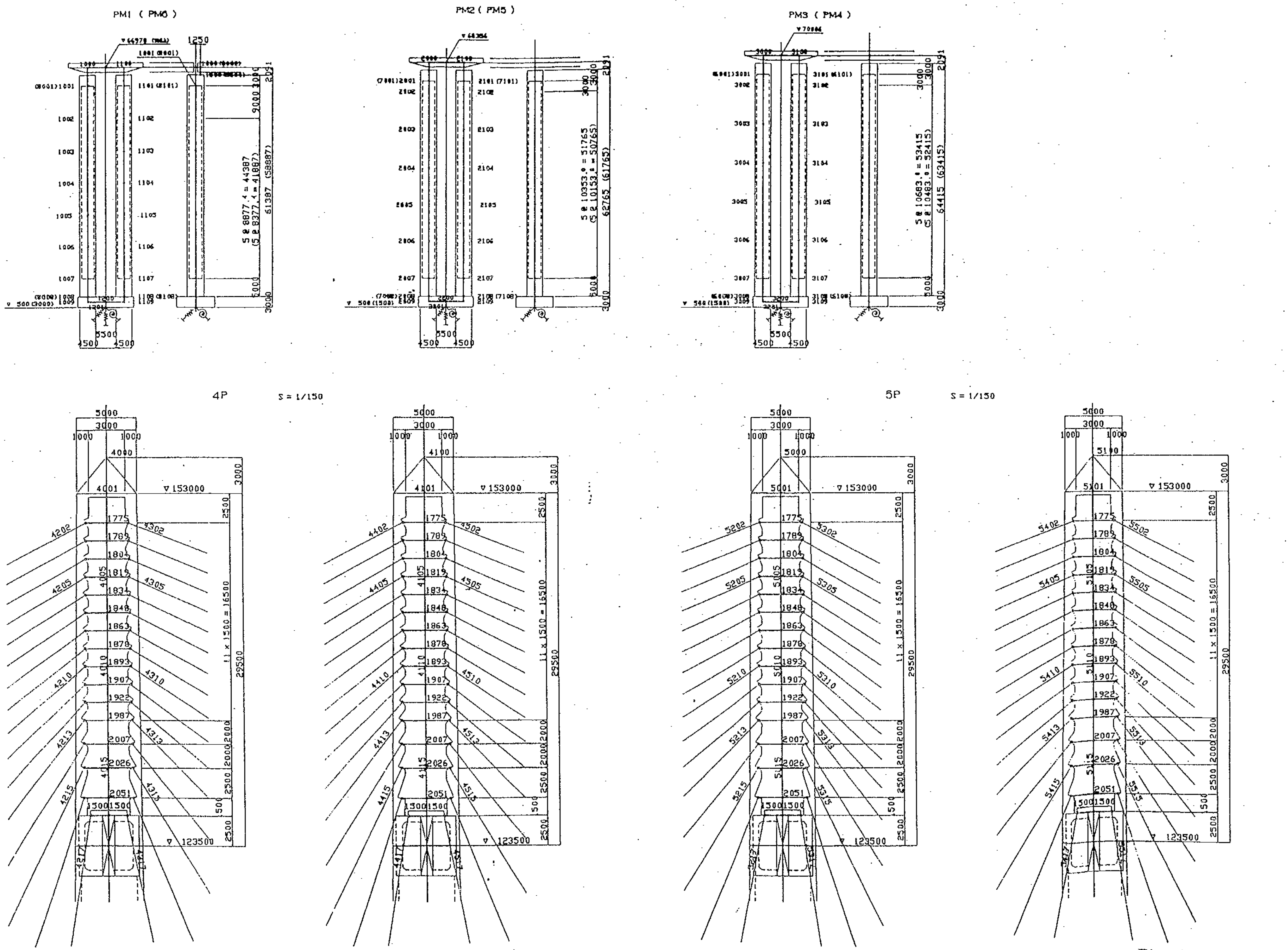


Fig. 4.1.5 Main Bridge Skelton (2)

Main Bridge Skelton (3) S = 1/500

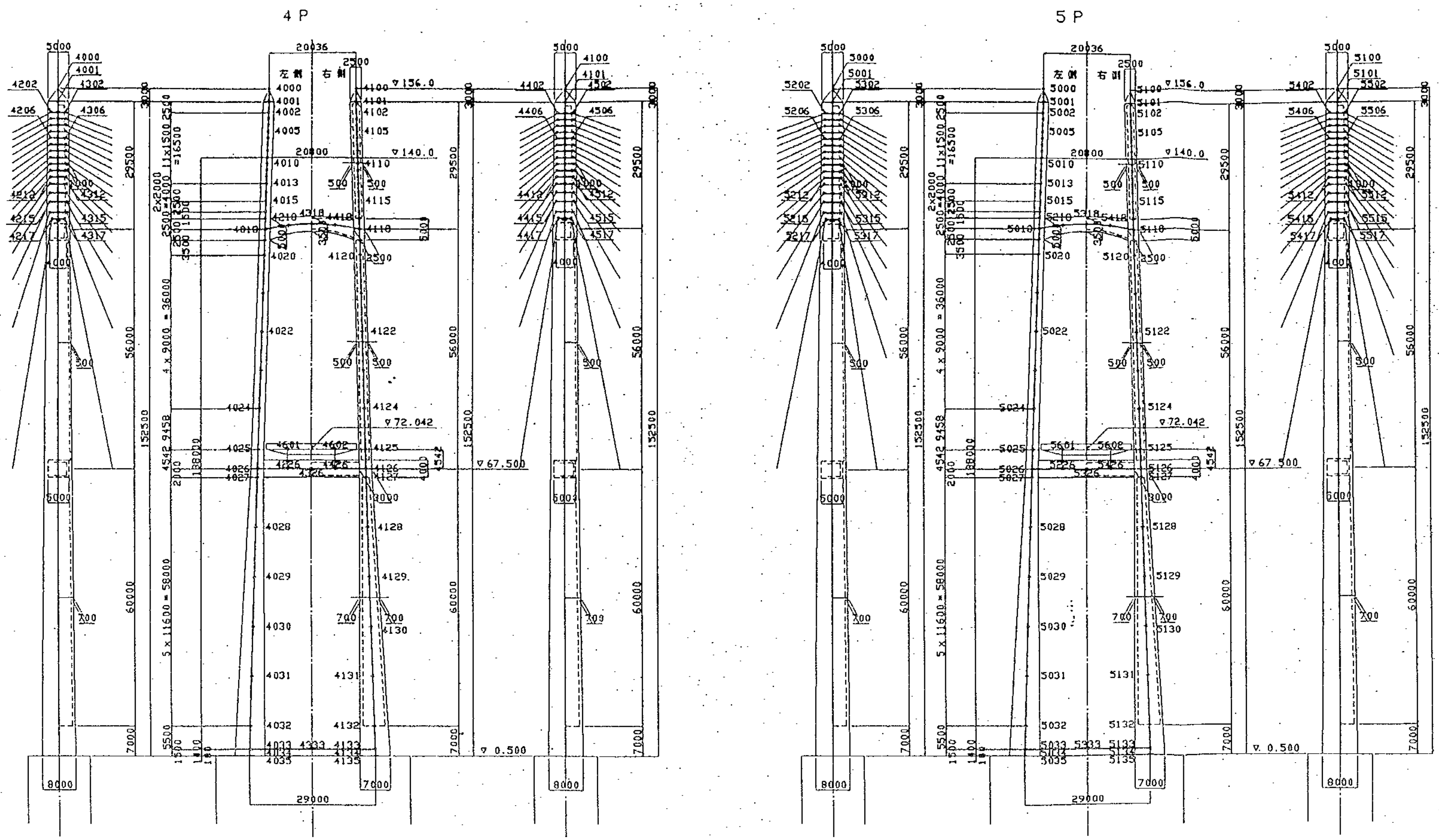


Fig. 4.1.6 Main Bridge Skelton (3)

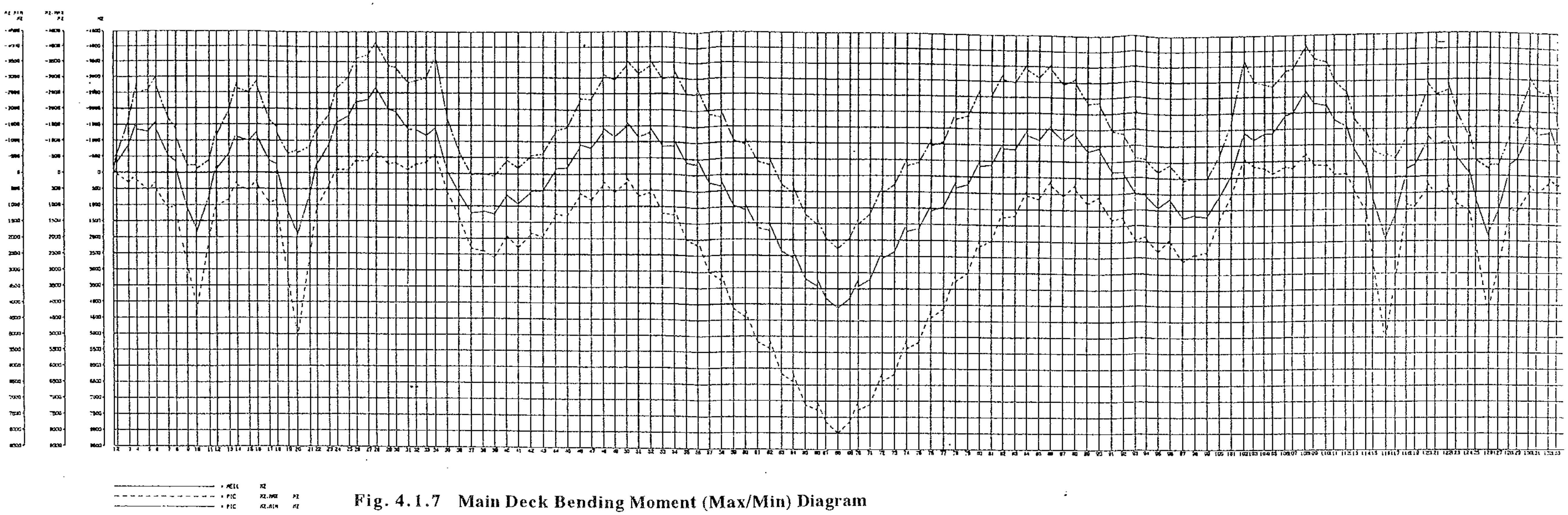


Fig. 4.1.7 Main Deck Bending Moment (Max/Min) Diagram

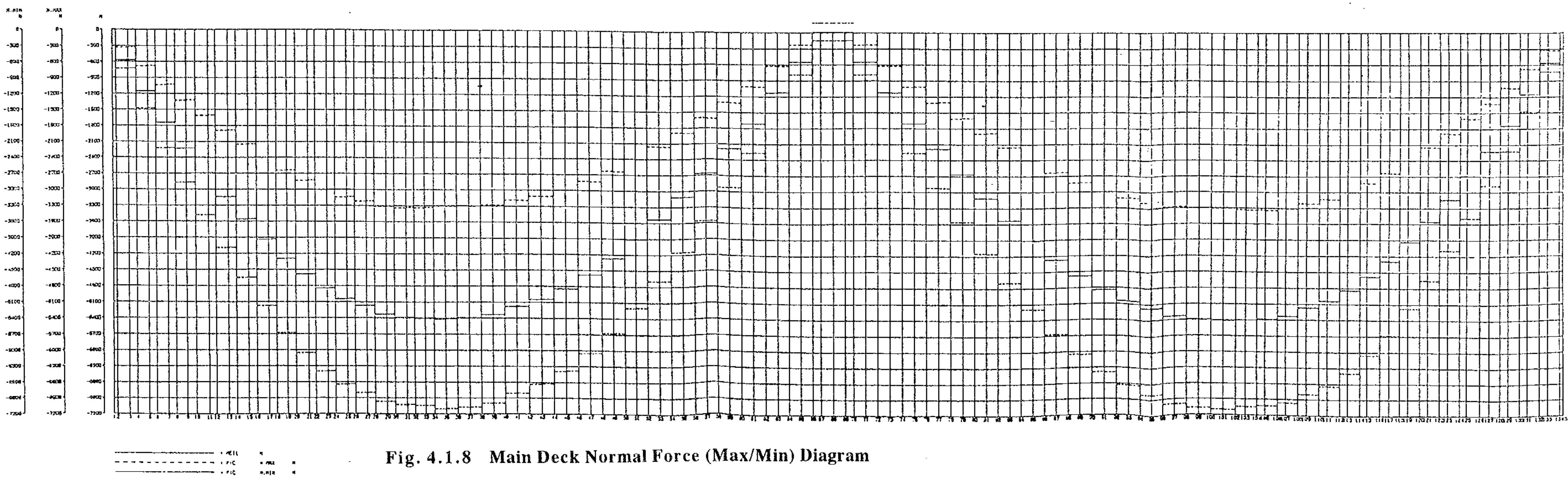


Fig. 4.1.8 Main Deck Normal Force (Max/Min) Diagram

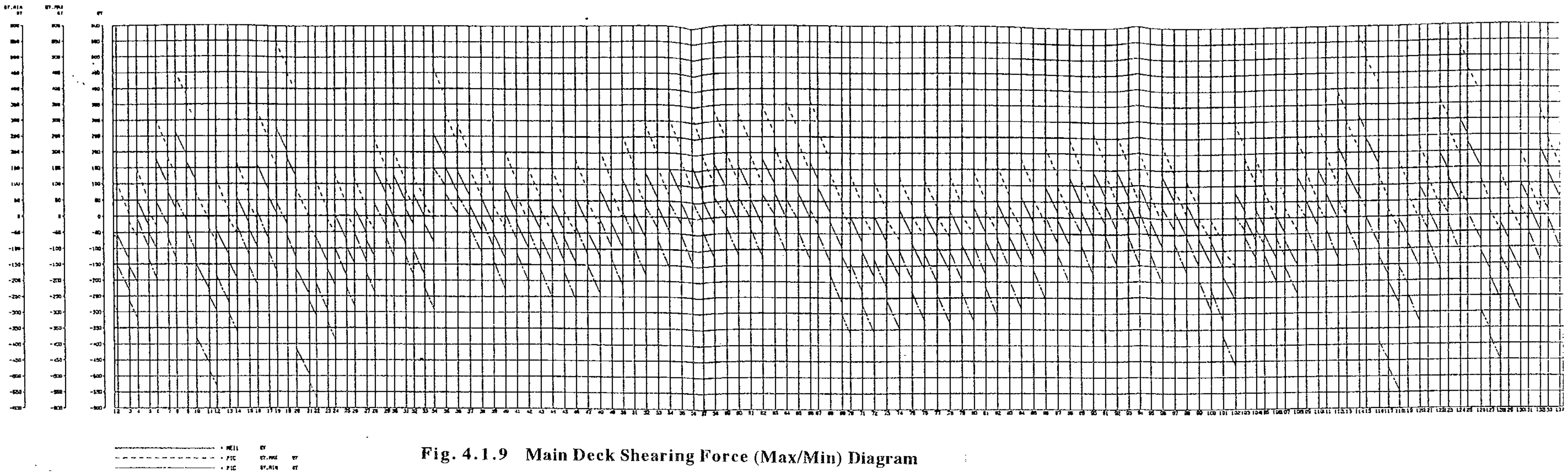


Fig. 4.1.9 Main Deck Shearing Force (Max/Min) Diagram

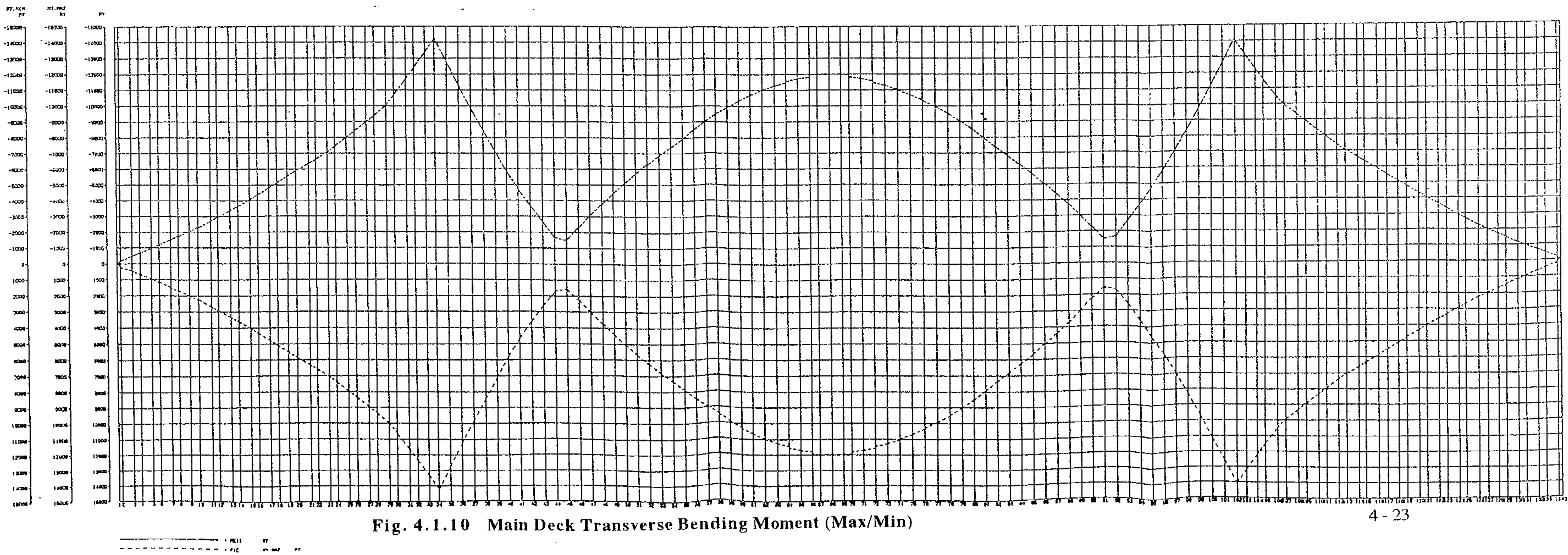


Fig. 4.1.10 Main Deck Transverse Bending Moment (Max/Min)

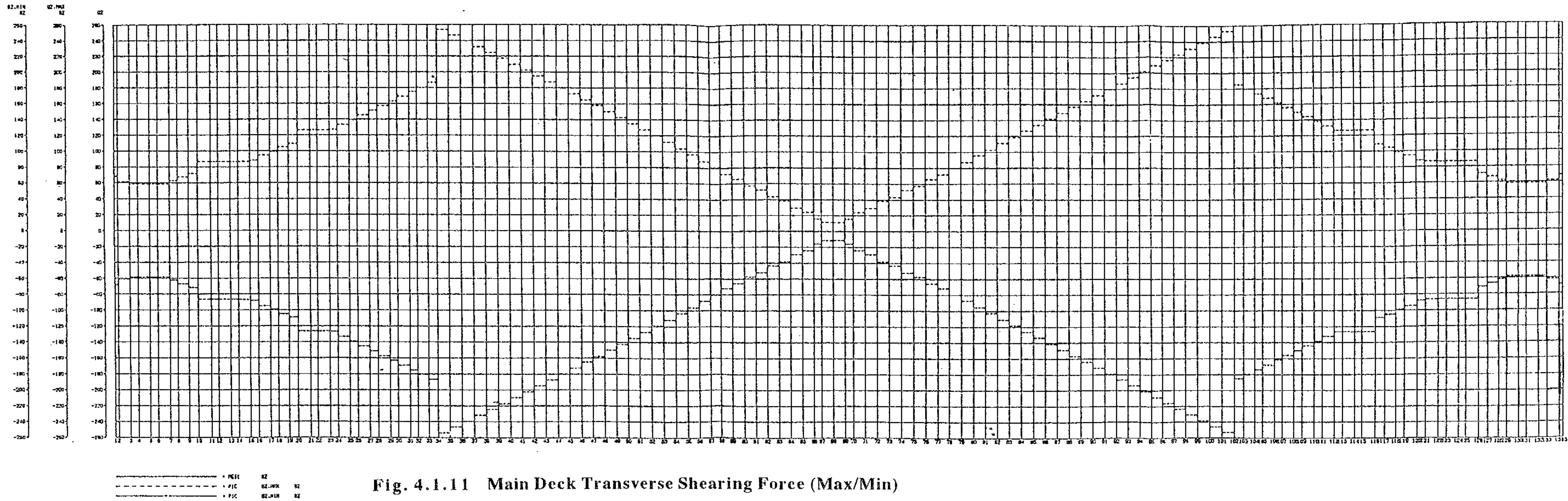
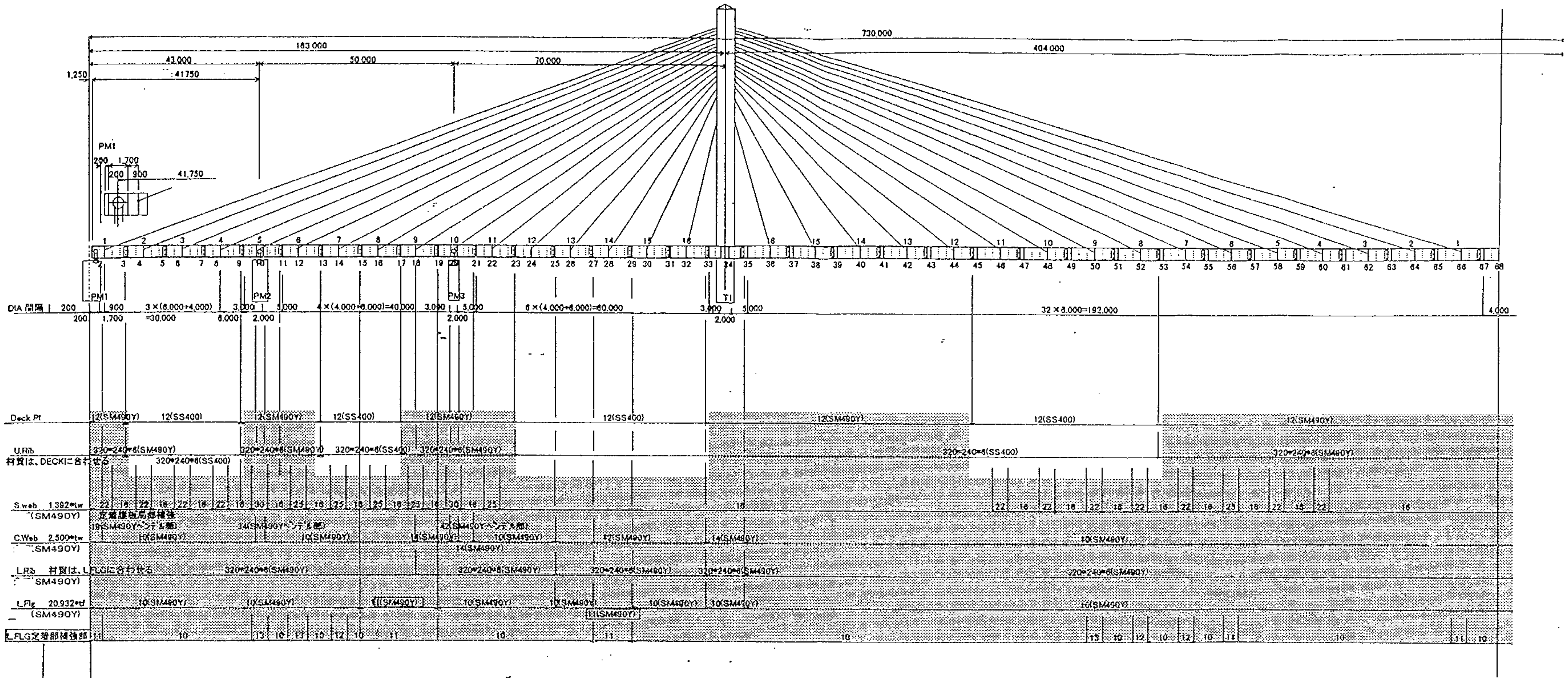
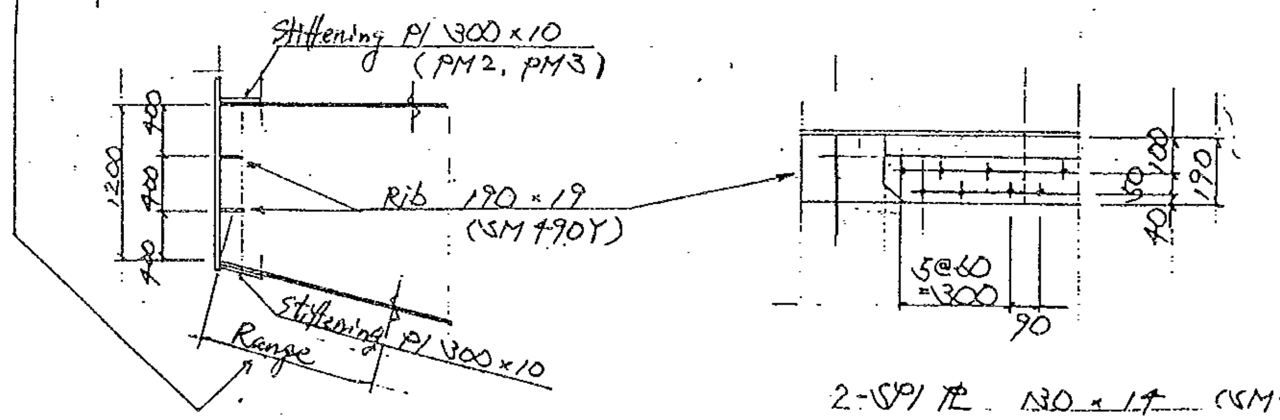


Fig. 4.1.11 Main Deck Transverse Shearing Force (Max/Min)

Main Deck Formation





2-SPl 材 130×14 (SM490Y)

12-HTB M22 (S10T)

- 1 -

Fig. 4.1.12 Main Deck Formation

Table 4.1.7 Girder Stresses of Major Sections (Deck Plate)

Section	Girder Effect (kg/cm^2)			Diaphragm Effect (kg/cm^2)			Transv. Rib Effect (kg/cm^2)		
	σ max.	σ min.	τ	σ y	τ	K< 1.2	σ y	τ	K< 1.2
5 (SS400)	205	-221	91	336	187	0.415	585	468	0.998
10 (SM490Y)	-225	-1310	198	264	399	0.917	585	468	0.974
13 (SS400)	-156	-508	160	336	187	0.691	585	468	1.036
20 (SM490Y)	-370	-1683	187	264	399	0.999	585	468	1.114
25 (SS400)	-42	-591	99	336	187	0.682	585	468	1.016
34 (SM490Y)	174	-750	279	435	481	0.803	585	468	0.832
45 (SS400)	-320	-758	136	336	187	0.802	585	468	0.916
57 (SM490Y)	64	-744	164	336	187	0.542	585	468	0.761
67 (SM490Y)	114	-960	187	336	187	0.636	585	468	0.844

Table 4.1.8 Girder Stresses of Major Sections (Lower Flg. Plate)

Section	Girder Effect (kg/cm^2)			Diaphragm Effect (kg/cm^2)			Transv. Rib Effect (kg/cm^2)		
	σ max.	σ min.	τ	σ y	τ	K< 1.2	σ y	τ	K< 1.2
5 (SM490Y)	-29	-688	76	-387	187	0.359	-728	384	0.511
10 (SM490Y)	802	-324	278	-363	478	0.799	-728	384	0.838
13 (SM490Y)	-204	-896	222	-387	187	0.504	-728	384	0.640
20 (SM490Y)	570	-635	262	-363	478	0.729	-728	384	0.760
25 (SM490Y)	-220	-1384	144	-387	187	0.650	-728	384	0.721
34 (SM490Y)	-9	-1685	299	-476	577	1.023	-728	384	0.900
45 (SM490Y)	-328	-1023	164	-387	187	0.517	-728	384	0.630
57 (SM490Y)	142	-717	175	-387	187	0.423	-728	384	0.579
67 (SM490Y)	1266	-334	191	-387	187	0.780	-728	384	0.960

$K = (\sigma/\sigma a)^2 - (\sigma/\sigma a)\cdot(\sigma y/\sigma a) + (\sigma y/\sigma a)^2 + (\tau/\tau a)^2$

Table 4.1.9 Girder Stress of Major Sections

Section	Deck Plate				U-Rib				Allowable Stress (kg/cm^2)	
	Girder Effect		Deck Effect		Girder Effect		Deck Effect		Sole	combi nation
	max	min	max	min	max	min	max	min		
5 (SS400)	205	- 221	460	- 445	123	- 204	1097	- 1134	1400	2000
10 (SM490Y)	- 225	- 1310	460	- 445	- 229	- 1116	1097	- 1134	2100	3000
13 (SS400)	- 156	- 508	460	- 445	- 185	- 480	1097	- 1134	1400	2000
20 (SM490Y)	-370	- 1683	460	- 445	- 380	- 1370	1097	- 1134	2100	3000
25 (SS400)	- 42	- 592	460	- 445	- 225	- 606	1097	- 1134	1400	2000
34 (SM490Y)	174	- 750	460	- 445	- 280	- 497	1097	- 1134	2100	3000
45 (SS400)	- 320	- 758	460	- 445	- 331	- 752	1097	- 1134	1400	2000
57 (SM490Y)	64	- 744	460	- 445	- 214	- 663	1097	- 1134	2100	3000
67 (SM490Y)	114	-960	460	- 445	- 203	- 756	1097	- 1134	2100	3000

4.1.6 Stay Cable

Stay cable arrangement is summarized in Table 4.1.10 as below;

Table 4.1.10 Stay Cable Arrangement

Cable No.	Cable Tension (ton)		Cable size	Allowable Tension (ton) 0.4Pu
	Side Span Side	Main Span Side		
1	433.8	428.0	48H15	510.8
2	444.5	438.5	48H15	510.8
3	453.3	443.7	48H15	510.8
4	401.2	389.2	48H15	510.8
5	403.4	385.8	48H15	510.8
6	394.3	372.0	48H15	510.8
7	378.9	354.9	48H15	510.8
8	370.2	340.9	48H15	510.8
9	370.3	333.4	48H15	510.8
10	320.0	284.8	37H15	393.6
11	397.3	267.1	37H15	393.6
12	234.9	213.8	27H15	287.2
13	190.9	178.5	27H15	287.2
14	231.8	216.5	27H15	287.2
15	117.0	113.4	19H15	202.0
16	78.7	75.0	19H15	202.0

4.1.7 Miscellaneous Analyses

(1) Secondary Stress Effects

1) Cable Sag

The cable spans between deck girder and pylon in the catenary shape due to its own weight. This catenary line will be influenced by the cable stress changes through vehicle loadings or other loadings. In the design work, this cable sag change is treated by reducing the nominal Young modulas of cables. The formula below is used for the estimation of this effect ;

$$E_1 = E_0 / (1 + g^2 * l^2 * E_0 / (12 * s_0^3)$$
$$E_2 = E_0 / (1 + g^2 * l^2 * (s_0 + s_1) * E_0 / (24 * s_0^2 * s_1^2))$$

where,

E_0 : Young modulas without cable sag
E_1 : Young modulas with cable sag at stress s_0
E_2 : Young modulas with cable sag between s_0 and s_1
g : unit weight of cable
l : horizontal projective length of cable
s_0 : cable stress at dead load condition
s_1 : cable stress at other loading condition

E_1 represents a tangential Young modulas and E_2 a secant Young modulas from initial stress level to new level. If s_1 is greater than s_0, then E_1 is to be less than E_2.

Table 4.1.11 and 4.1.12 represents the result of E_1 values of the cables at dead load and prestressing condition.

In order to assess the influence range of stress and deformation of the bridge members, the analysis is carried out using E_1 values instead of E_0 for the Young modulas of the cables. This will give the conservative range of influence of cable sag.

Table 4.1.13 shows the result of live load deformation of the major points of the bridge. Table 4.1.14 shows the comparison of main girder bending moment. The difference of bending moment is within some 5% of estimation by E_0.

Table 4.1.11 E_1 value of the Cable (Side Span side)

No.	S_t (t/m^2)	E_0 (t/m^2)	g (t/m^3)	l (m)	E_1 (t/m^2)
1	48273	20000000	15.00	159.75	18430000
2	49835	20000000	15.00	149.75	18730000
3	50736	20000000	15.00	139.75	18930000
4	42342	20000000	15.00	129.75	18460000
5	42252	20000000	15.00	119.75	18460000
6	40646	20000000	15.00	109.75	18740000
7	38303	20000000	15.00	99.75	18750000
8	36351	20000000	15.00	89.75	18820000
9	35330	20000000	15.00	79.75	18970000
10	41404	20000000	13.84	69.75	19570000
11	37466	20000000	13.84	59.75	19580000
12	42827	20000000	13.52	49.75	19810000
13	32267	20000000	13.52	39.75	19720000
14	45493	20000000	13.52	29.75	19940000
15	31780	20000000	16.63	19.75	19890000
16	23333	20000000	16.63	9.75	19990000

Table 4.1.12 E_1 value of the Cable (Center Span side)

No.	S_1 (t/m²)	E_0 (t/m²)	g (t/m³)	l (m)	E_1 (t/m²)
1	45465	20000000	15.00	190.50	17470000
2	46832	20000000	15.00	178.50	17920000
3	47613	20000000	15.00	166.50	18240000
4	39610	20000000	15.00	154.50	17480000
5	39444	20000000	15.00	142.50	17790000
6	37823	20000000	15.00	130.50	17890000
7	35586	20000000	15.00	118.50	17910000
8	33709	20000000	15.00	106.50	18000000
9	32643	20000000	15.00	94.50	18240000
10	37797	20000000	13.84	82.50	19230000
11	33762	20000000	13.84	70.50	19210000
12	37947	20000000	13.52	58.50	19630000
13	28213	20000000	13.52	46.50	19430000
14	38960	20000000	13.52	34.50	19880000
15	27045	20000000	16.63	22.50	19770000
16	19508	20000000	16.63	10.50	19860000

Table 4.1.13 Comparison of Girder Vertical Deflection (Live Load)

Section	Deflection by E_0 (cm)	Deflection by E_1 (cm)	Ratio	Remarks
5	2.0	2.0	1.00	
15	2.1	2.2	1.01	
27	3.5	3.6	1.01	
40	12.5	12.8	1.03	
49	29.5	31.3	1.05	
58	54.0	57.8	1.07	
68	71.5	76.5	1.07	

Table 4.1.14 Comparison of Girder Bending Moment (Live Load)

Section	Bending M. by E_0 (cm)	Bending M. by E_1 (cm)	Ratio	Remarks
5	1558 / -1112	1564 / -1110	1.004	
15	1414 / -1430	1424 / -1425	1.007	
20	2422 / -1824	2475 / -1831	1.022	
27	1443 / -1275	1473 / -1334	1.046	
34	78 / -2175	81 / -2181	1.038	
40	1228 / -896	1233 / -951	1.061	
49	1597 / -1731	1636 / -1849	1.068	
58	2582 / -1879	2710 / -1918	1.050	
68	3781 / -541	3956 / -553	1.046	

2) Buckling of Concrete Wall

Pylon column has hollow box section. At deck girder elevation, its dimension is about 6.53 x 2.93 m and 0.5 m wall thickness. Local buckling of concrete wall can be checked by following formula (for steel plate under compression stress by Japanese road bridge design code) ;

$R <= 0.7 \quad s_{cr} / s_y = 1.0$

$R > 0.7 \quad s_{cr} / s_y = 0.5 / R^2$

where,

$R = b/t\,(s_y/E * 12 * (1-m^2)/p^2 * k)^{1/2} = 0.17$

s_{cr} : local buckling stress

s_y : yield stress (300 kg/cm^2 x 0.8 = 240 kg/cm^2)

E : Young modulas (2.75 x 10^5 kg/cm^2)

m : poisson ratio (0.3)

p : 3.1416

k : buckling factor (both ends restrained ; 4.0)

b : wall width (5.53 m)

t : wall thickness (0.5 m)

R value is far little compared with 0.7 and then local buckling is considered not to occure until concrete yield stress level.

Same calculation for the column base part gives the following result ;

box dimention : 7.86 x 6.58 m, wall thickness 0.7 m

R = 0.14

(2) Wind Stability

In order to certificate the wind stability of the deck girder, a series of wind tunnel test was carried out using the rigid two dimentional scale model (scale 1/40). Details of the tests procedure and results shall be refered to the separate report of wind tunnel test.

1) Test Arrangement

The models were prepared for the following test conditions;

girder	: depth = 2.5 and 3.0 m
guard net	: height = 2.25 and 2.75 m
gurd rail type	: rigid beam, guard rail and concrete wall
fender	: with and without cases
wind speed	: 0 to 52 m/s (actual scale)

2) Test Results

The test results for girder depth of 2.5 m are summarized in Table 4.1.15 and 4.1.16. From this test results, two alternatives of guard rail type (with guard net H=2.25m, and with fender) and rigid beam type (with guard net H=2.25m, and without fender) are considered to be stable to wind blows. Of these two alternatives, rigid beam type without fender is somewhat questionable from view point of traffic safety of maintenance workers on sidewalk.

Same tests were carried out for the models of girder depth of 3.0 m. The results unacceptable amplitude of vertical deflection by vortex excitation. Therefore, guard rail type alternative with girder depth of 2.5 m is recommended for the deck girder shape.

Table 4.1.15 Summary of Results (2.5 m Girder Depth, Deflection)

Case	Guard Rail Type	Fender	Attack Angle (deg)	Vortex Excitation (actual bridge level)				Flutter Excitation Wind Speed (m/s)
				wind speed (m/s)	ampli tude (cm)	revised amp. (cm)	allowab le Amp. (cm)	
1	Rigid Beam, Guard Net H=2.25m	with	0	no	-	-	10.5	not appeared
2			3	16.1	18.4	23.9		not appeared
3			-3	no	-	-		not appeared
4	Guard Rail, Guard Net H=2.25m	with	0	no	-	-		not appeared
5			3	17.3	5.8	7.6		not appeared
6			-3	no	-	-		not appeared
7	Concrete Wall, Guard Net H=2.25m	with	0	16.8	32.4	42.1		not appeared
8			3	16.3	38.2	49.7		not appeared
9			-3	19.8	28.1	36.6		not appeared
10	Rigid Beam, Guard Net H=2.75m	with	0	no	-	-		not appeared
11			3	17.0	14.2	18.5		not appeared
12			-3	no	-	-		not appeared
13	Rigid Beam, Guard Net H=2.25m	without	0	8.7	1.4	1.9		not appeared
14			3	9.2	1.3	1.8		not appeared
15			-3	no	-	-		not appeared
16	Guard Rail, Guard Net H=2.25m	without	0	no	-	-		not appeared
17			3	18.6	13.0	16.8		not appeared
18			-3	no	-	-		not appeared

note : revised factor = 1.3

Table 4.1.16 Summary of Results (2.5 m Girder Depth, Tortion)

Case	Guard Rail Type	Fender	Attack Angle (deg)	Vortex Excitation (actual bridge level)				Flutter Excitation Wind Speed (m/s)
				wind speed (m/s)	amplitude (deg)	revised amp. (deg)	allowable Amp. (deg)	
1	Rigid Beam, Guard Net H=2.25m	with	0	no	-	-	0.33	not appeared
2			3	20.6	0.06	0.08		not appeared
3			-3	no	-	-		not appeared
4	Guard Rail, Guard Net H=2.25m	with	0	no	-	-	0.31	not appeared
5			3	no	-	-		not appeared
6			-3	no	-	-		not appeared
7	Concrete Wall, Guard Net H=2.25m	with	0	18.9	0.61	0.79	0.33	not appeared
8			3	19.6	0.53	0.69		not appeared
9			-3	21.7	0.63	0.83		not appeared
10	Rigid Beam, Guard Net H=2.75m	with	0	no	-	-		not appeared
11			3	no	-	-		not appeared
12			-3	no	-	-		not appeared
13	Rigid Beam, Guard Net H=2.25m	without	0	no	-	-		not appeared
14			3	no	-	-		not appeared
15			-3	no	-	-		not appeared
16	Guard Rail, Guard Net H=2.25m	without	0	no	-	-	0.31	not appeared
17			3	25.1	0.07	0.08		not appeared
18			-3	no	-	-		not appeared

note : revised factor = 1.3

(3) Dualability Check by BS5400

1) Load combination

Following load combinations are checked based on the BS 5400;

Table 4.1.17 Load Combination and Load Factors

Combination Load	1		2	3	E.Q.
	ULS	SLS	ULS	ULS	ULS
Girder Weight	1.05	1.00	1.05	1.05	1.05
Substructer Weight	1.15	1.00	1.15	1.15	1.15
Superimposed D. L.	1.40	1.10	1.40	1.40	1.40
Defferential Settlement	1.20	1.00	1.20	1.20	1.20
Temperature Change				1.30	
Wind Load			1.10		
Earthquake					1.30
Live Load	1.50	1.20	1.25	1.25	

2) Results of Analysis

The results of stress conditions of major sections of the Bridge structure are summarized as below (details shall be refered to separate report);

a) Pylon Leg

Two sections of leg base and below deck girder are reviewed. The acting forces used are shown in Table 4.1.18.

* leg base

- N max : Mx = 53494 KN. m, My = 214326 KN. m
  N capacity = 377956 KN > 143886 KN
- My max : Mx = 19176 KN. m, N = 132325 KN
  My capacity = 682196 KN. m > 412335 KN. m
- Mx max : My = 2742 KN. m, N = 105590 KN
  Mx capacity = 572312 KN. m < 666516 KN. m
  but Gfl = 1.3 used. By AASHTO Gfl = 1.0,
  capacity is adequate.

* below deck girder

- My max : Mx = 67626 KN. m,   N = 111540 KN
  My capacity = 149972 KN. m > 148563 KN. m
- Mx max : My = 34123 KN. m,   N = 83942 KN
  Mx capacity = 92774 KN. m > 65617 KN. m

Table 4.1.18 Check Force Combination for Pylon Leg

NODE 4032 PYLON LEG AT BASE

CASE		MY	QZ	N	MX	QY	T
COMB EQ MY=MAX (MIN)	(tonnes and metres)	38211	983.3	-12262.5	-1531.8	15.4	249.8
	(kN and metres)	412334.9	10610.79	-132324.6	-16529.65	166.1814	2695.5918
COMB EQ QZ = MAX (MIN)	(tonnes and metres)	-380466.6	-1008.7	-7442.6	-903.6	16.5	-268.9
	(kN and metres)	-410560.9	-10884.88	-80313.1	-9750.748	178.0515	-2901.7
COMB 2N = MIN	(tonnes and metres)	19594.9	-491.8	-13333.9	4690.6	19.9	-491.4
	(kN and metres)	211448.57	-5307.014	-143886.1	50616.265	214.7409	-5302.697
COMB EQ MX=MAX (MIN)	(tonnes and metres)	58.4	-13.6	-9785.1	61765.9	1078.6	281.9
	(kN and metres)	630.1944	-146.7576	-105591	666515.3	11639.173	3041.9829
COMB EQ QY=MAX (MIN)	(tonnes and metres)	58.4	-13.6	-9785.1	61765.9	1078.6	281.9
	(kN and metres)	630.1944	-146.7576	-105591	666515.83	11639.13	3041.9829
COMB 2T MIN (MAX)	(tonnes and metres)	-6833.4	-387.4	-10650.3	2189.7	24.5	-1023
	(kN and metres)	-73739.22	-4180.433	-114927.4	23629.053	264.3795	-11039.19

NODE 4027 PYLON LEG ~~AT BASE~~ BELOW LEG

CASE		MY	QZ	N	MX	QY	T
COMB EQ MY=MAX (MIN)	(tonnes and metres)	-13767.3	815.1	-10338.6	-197.7	5.9	226.3
	(kN and metres)	-148562.9	8795.7441	-11563.8	-2133.381	63.6669	2442.0033
COMB EQ QZ = MAX (MIN)	(tonnes and metres)	-13574.6	815.1	10249.6	844	15.4	249.2
	(kN and metres)	-146483.5	8795.7441	110603.43	9107.604	166.1814	2689.1172
COMB 2N = MIN	(tonnes and metres)	-7690.2	-352.6	-11325.3	3497.1	19.9	-491.4
	(kN and metres)	-82984.95	-3804.907	-122211.3	37737.206	214.7409	-5302.697
COMB EQ MX=MAX (MIN)	(tonnes and metres)	-921.9	45.3	-7778.9	6080.7	851.7	281.8
	(kN and metres)	-9948.223	488.8323	-83942.11	65616.84	9190.6947	3040.9038
COMB EQ QY=MAX (MIN)	(tonnes and metres)	-921.9	45.3	-7778.9	6080.7	851.7	2818
	(kN and metres)	-9948.223	488.8323	-83942.11	65616.834	91903.697	3040.9038
COMB 2T MIN (MAX)	(tonnes and metres)	-1007.9	47.2	-7913.7	5501	-819.7	281.8
	(kN and metres)	-10876.25	509.3352	-85396.74	59361.291	-8845.383	3040.908

b) Steel Deck

Three sections of deck (node 34,50 and 68) are checked;

Table 4.1.19 Deck Stress and Allowable Stress

Section	Applied		Area (mm^2)	Sectional Modulus ($E+08\ mm^3$)		Stress (N/mm^2)		Allowable Stress (N/mm^2)
	N(KN)	M(KN.m)		top	bottom	top	bottm	
Node 68								
N max	-2329.9	101034.2	808600	7.7857	-6.0225	126.9	-170.6	212
M max	-418.9	105142.6	808600	7.7857	-6.0225	134.5	-175.1	212
Node 50								
N max	59298.5	-46171.7	808600	7.7857	-6.0225	14.0	150.0	212
M max	59298.5	-46171.7	808600	7.7857	-6.0225	14.0	150.0	212
Node 34								
N max	82383.4	-40969.5	651410	5.9486	-4.5648	57.6	216.2	314
M max	76319.8	-56513.4	651410	5.9486	-4.5648	22.2	241.0	314

Note : Allowable stress : SS400 = 245 N/mm^2 / 1.1 * 1.05 = 212 N/mm^2
SM490Y = 363 N/mm^2 / 1.1 * 1.05 = 341 N/mm^2

c) Stay Cable

Table 4.1.19 shows the results of stress level under SLS at combination 1 of stay cable. Maximum stress level is 83% of the cable capacity.

Table 4.1.20 Stay Cable Stress Result

cable ref	no of strands	stay GUTS	GUTS/2.3	sls comb 1	% capacity	cable ref	no of strands	stay GUTS	GUTS/2.3	sls comb 1	% capacity
602 -4202	48	12720	5530.4	4396.4	79%	802 -4402	48	12720	5530.4	4396.4	79%
604 -4203	48	12720	5530.4	4504.1	81%	804 -4403	48	12720	5530.4	4504.1	81%
606 -4204	48	12720	5530.4	4595.9	83%	806 -4404	48	12720	5530.4	4595.9	83%
608 -4205	48	12720	5530.4	4092.8	74%	808 -4405	48	12720	5530.4	4092.8	74%
610 -4206	48	12720	5530.4	4119.8	74%	810 -4406	48	12720	5530.4	4119.8	74%
612 -4207	48	12720	5530.4	4037.4	73%	812 -4407	48	12720	5530.4	4037.4	73%
614 -4208	48	12720	5530.4	3893.1	70%	814 -4408	48	12720	5530.4	3893.1	70%
616 -4209	48	12720	5530.4	3816.1	69%	816 -4409	48	12720	5530.4	3816.1	69%
618 -4210	48	12720	5530.4	3827.5	69%	818 -4410	48	12720	5530.4	3827.5	69%
620 -4211	37	9805	4263.0	3295.5	77%	820 -4411	37	9805	4263.0	3295.5	77%
622 -4212	37	9805	4263.0	3076.6	72%	822 -4412	37	9805	4263.0	3076.6	72%
624 -4213	27	7155	3110.9	2428.1	78%	824 -4413	27	7155	3110.9	2428.1	78%
626 -4214	27	7155	3110.9	2002.7	64%	826 -4414	27	7155	3110.9	2002.7	64%
628 -4215	27	7155	3110.9	2400.0	77%	828 -4415	27	7155	3110.6	2400.0	77%
630 -4216	19	5035	2189.1	1223.0	56%	830 -4416	19	5035	2189.1	1223.0	56%
632 -4217	19	5035	2189.1	819.7	37%	832 -4417	19	5035	2189.1	819.7	37%
666 -4302	48	12720	5530.4	4250.2	77%	866 -4402	48	12720	5530.4	4250.2	77%
664 -4303	48	12720	5530.4	4449.9	80%	864 -4403	48	12720	5530.4	4449.9	80%
662 -4304	48	12720	5530.4	.4501.4	81%	862 -4404	48	12720	5530.4	4501.4	81%
660 -4305	48	12720	5530.4	3972.7	72%	860 -4405	48	12720	5530.4	3972.7	72%
658 -4306	48	12720	5530.4	3940.0	71%	858 -4406	48	12720	5530.4	3940.0	71%
656 -4307	48	12720	5530.4	3813.0	69%	856 -4407	48	12720	5530.4	3813.0	69%
654 -4308	48	12720	5530.4	3655.3	66%	854 -4408	48	12720	5530.4	3655.3	66%
652 -4309	48	12720	5530.4	3531.8	64%	852 -4409	48	12720	5530.4	3531.8	64%
650 -4310	48	12720	5530.4	3477.9	63%	850 -4410	48	12720	5530.4	3477.9	63%
648 -4311	37	9805	4263.0	2973.4	70%	848 -4411	37	9805	4263.0	2973.4	70%
646 -4312	37	9805	4263.0	2823.3	66%	846 -4412	37	9805	4263.0	2823.3	66%
644 -4313	27	7155	3110.9	2263.6	73%	844 -4413	27	7155	3110.9	2263.6	73%
642 -4314	27	7155	3110.9	1930.7	62%	842 -4414	27	7155	3110.9	1930.7	62%
640 -4315	27	7155	3110.9	2303.0	74%	840 -4415	27	7155	3110.9	2303.0	74%
638 -4316	19	5035	2189.1	1222.4	56%	838 -4416	19	5035	2189.1	1222.4	56%
636 -4317	19	5035	2189.1	805.6	37%	836 -4417	19	5035	2189.1	805.6	37%
				maximum	83%					maximum	83%

GUTS strand 265 kN

4.2 Calculation Analysis for Approach Bridges

4.2.1 General Description

The structural calculation method for the detailed design will basically follow the "Allowable Stress Design (service load design) Method" in accordance with "Specifications for Highway Bridge" practiced in Japan. However, prestressed concrete structures will be so designed as to ensure their safety in the ultimate loading condition prescribed in these specifications.

Bridges to be designed in this detailed design study are listed in Table 4.2.1 and general view of the approach bridges are shown in Figure 4.2.1.

Table 4.2.1 Summary of bridge length and span arrangement

Location	Bridge Designation	Structure Type	Bridge Length and Span Arrangement
The West Bank	BREW	5 spans continuous PC box girder	5 @40000 = 200000
	BRW2	4 spans continuous PC box girder	30500+40000+40000+30500 = 141000
	BRW3	5 spans continuous PC box girder	5 @37800 = 189000
	BRW4	4 spans continuous PC box girder	30500+40000+40000+30500 = 141000
	BRW5	6 spans continuous PC rigid frame	6 @36600 = 219600
	BRW6	7 spans continuous PC rigid frame	7 @40000 = 280000
	BRW7	7 spans continuous PC rigid frame	7 @40000 = 280000
	BRW8	7 spans continuous PC rigid frame	7 @40000 = 280000
The East Bank	BRE1	5 spans continuous PC rigid frame	5 @40000 = 200000
	BRE2	5 spans continuous PC rigid frame	5 @40000 = 200000
	BRE3	5 spans continuous PC rigid frame	5 @40000 = 200000
	BRE4	7 spans continuous PC rigid frame	7 @40000 = 280000
	BRE5	7 spans continuous PC rigid frame	7 @40000 = 280000
	BRE6	7 spans continuous PC rigid frame	7 @40000 = 280000

PC rigid frame means that the structure comprising the superstructure of prestressed concrete and the substructure of reinforced concrete are rigidly connected, without bridge bearings.

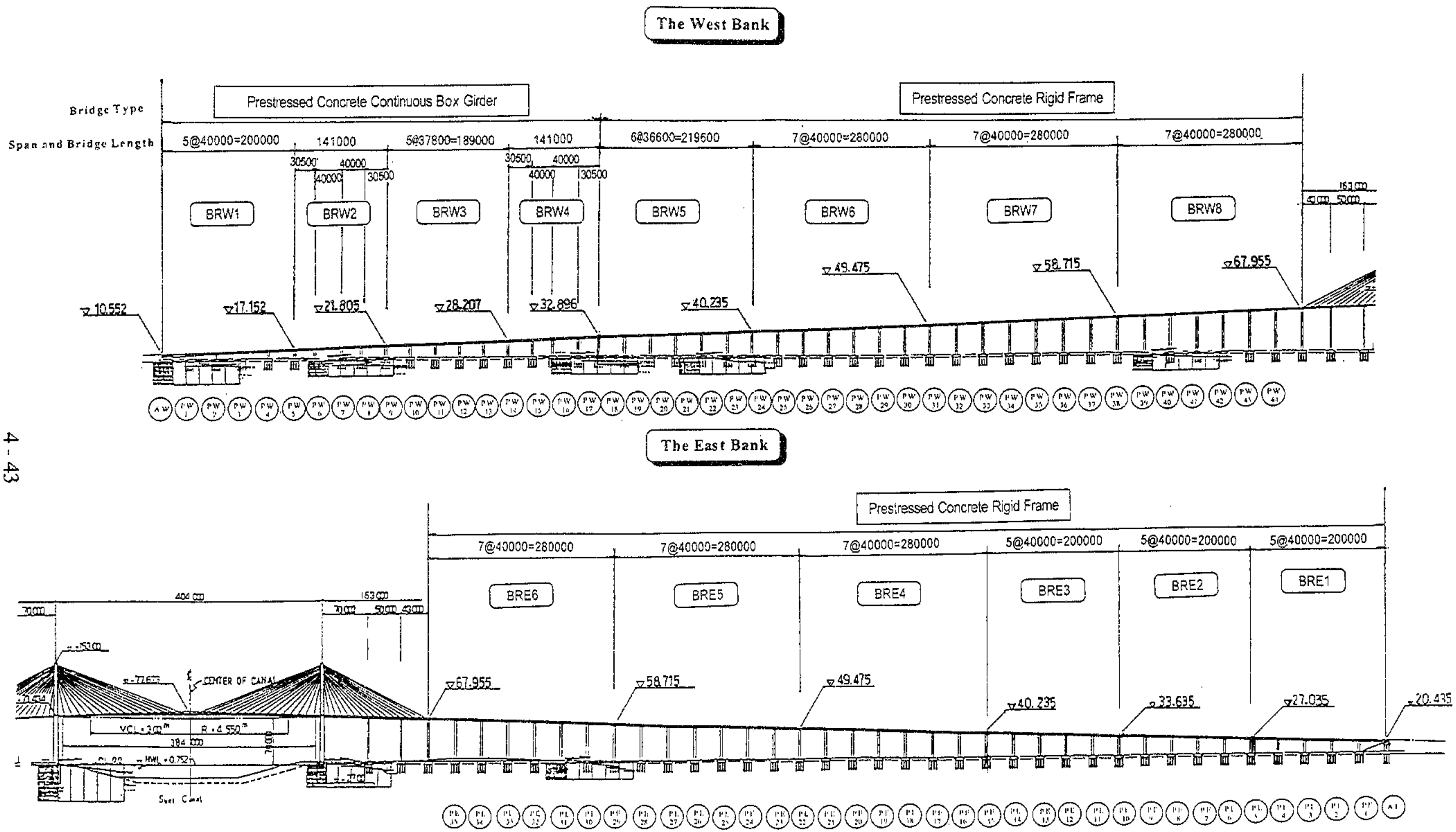


Fig. 4.2.1 General View of Suez Canal Approach Bridges

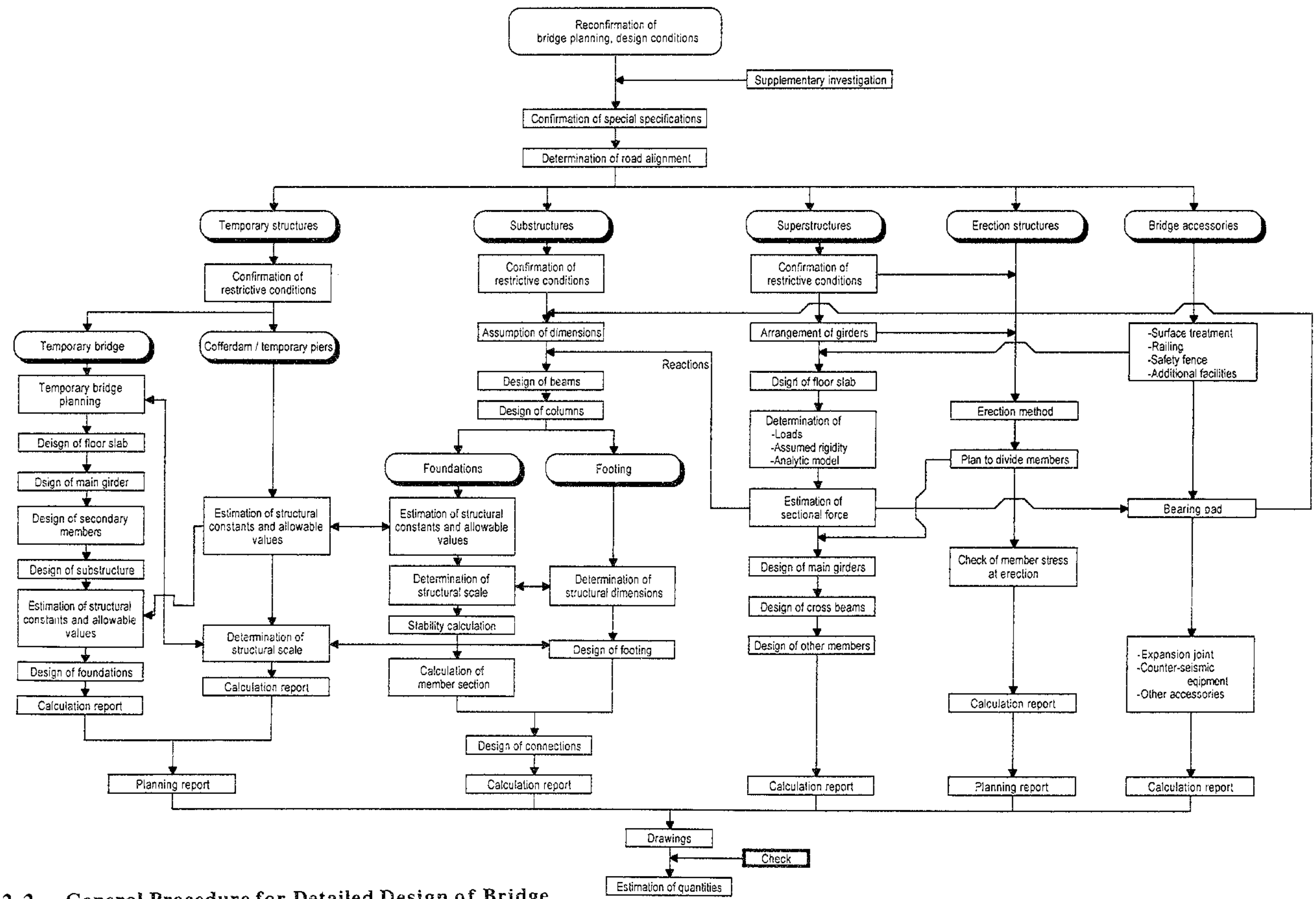


Fig. 4.2.2 General Procedure for Detailed Design of Bridge

4.2.2 Foundations

(1) General Conditions

Bearing stratum for the bridge foundation, which has Standard Penetration Test value (N value), greater than 50 is located around 5m to 7m below the ground surface.

(2) Type of Foundation

Pile foundations have been adopted for all piers and abutments of the Suez Canal approach bridges, and are as follows:

Type of pile : Cast-in-place reinforced concrete pile
Pile diameter : ϕ 1500 mm
Construction method : All casing method
Pile length : L=15 m

(3) Bearing Capacity of Pile

In Egypt, bearing capacity per pile is calculated on the condition that friction between pile and soil is ignored. Therefore, in this detailed design, bearing capacity per pile under ordinary load condition is calculated on the basis of qa = 280 tf/m^2, which is allowable bearing capacity for a 1500 mm diameter pile.

Table 4.2.2 Bearing Capacity of Pile

Normal condition	Allowable bearing capacity	494.76 tf/pile
Seismic condition	Allowable bearing capacity	742.14 tf/pile

(4) Method of Calculation

The pile reaction and the amount of displacement are calculated by an elastic analysis method taking displacement into account (the displacement method).

The displacement method is based on the following condition:

a) Two dimensional analysis
b) Footing is considered as a rigid body and the rotational center is the center of piles
c) Pile is considered as a elastic body
d) Soil is considered as a spring (soil spring)

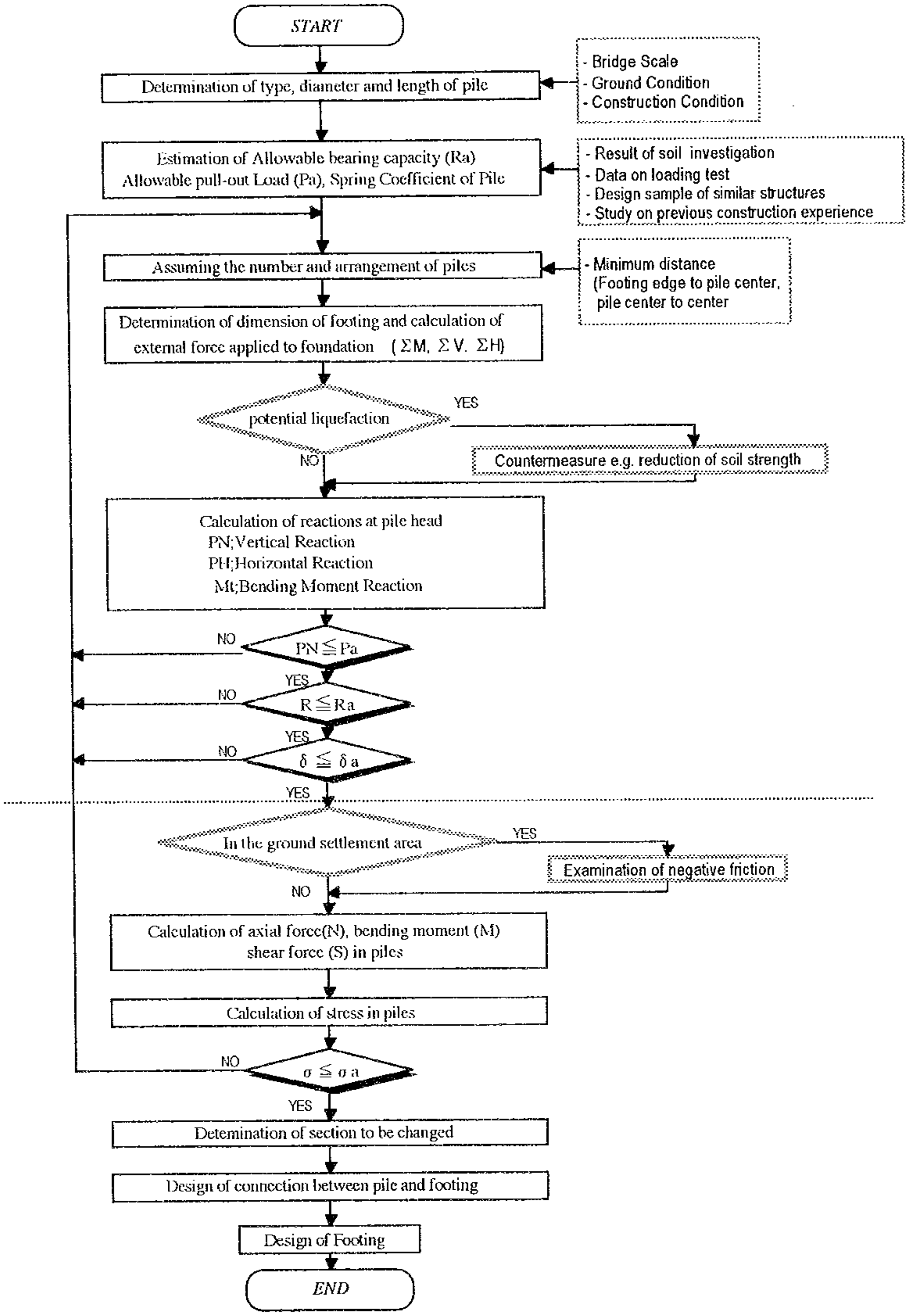


Fig. 4.2.3 General Procedure for Design of Bridge Foundations

4.2.3 Substructures

(1) General Condition

Structural type of substructures for the approach bridges are shown in Figure 4.2.4.

Abutment : Reinforced concrete abutment , Inverted T type
Pier : Reinforced concrete pier, type of hollow rectangular section

Piers are to be constructed as hollow section, structures but the top and the bottom of the pier are to be solid sections. The full section at the bottom is needed to resist large sectional forces and the full section at the top to permit the installation of brackets to support the girder of the Movable Scaffolding. However, the shorter piers have no hollow section which will simplify their construction.

Piers and abutments of the approach bridges with their properties are listed in Tables 4.2.3 and 4.2.4.

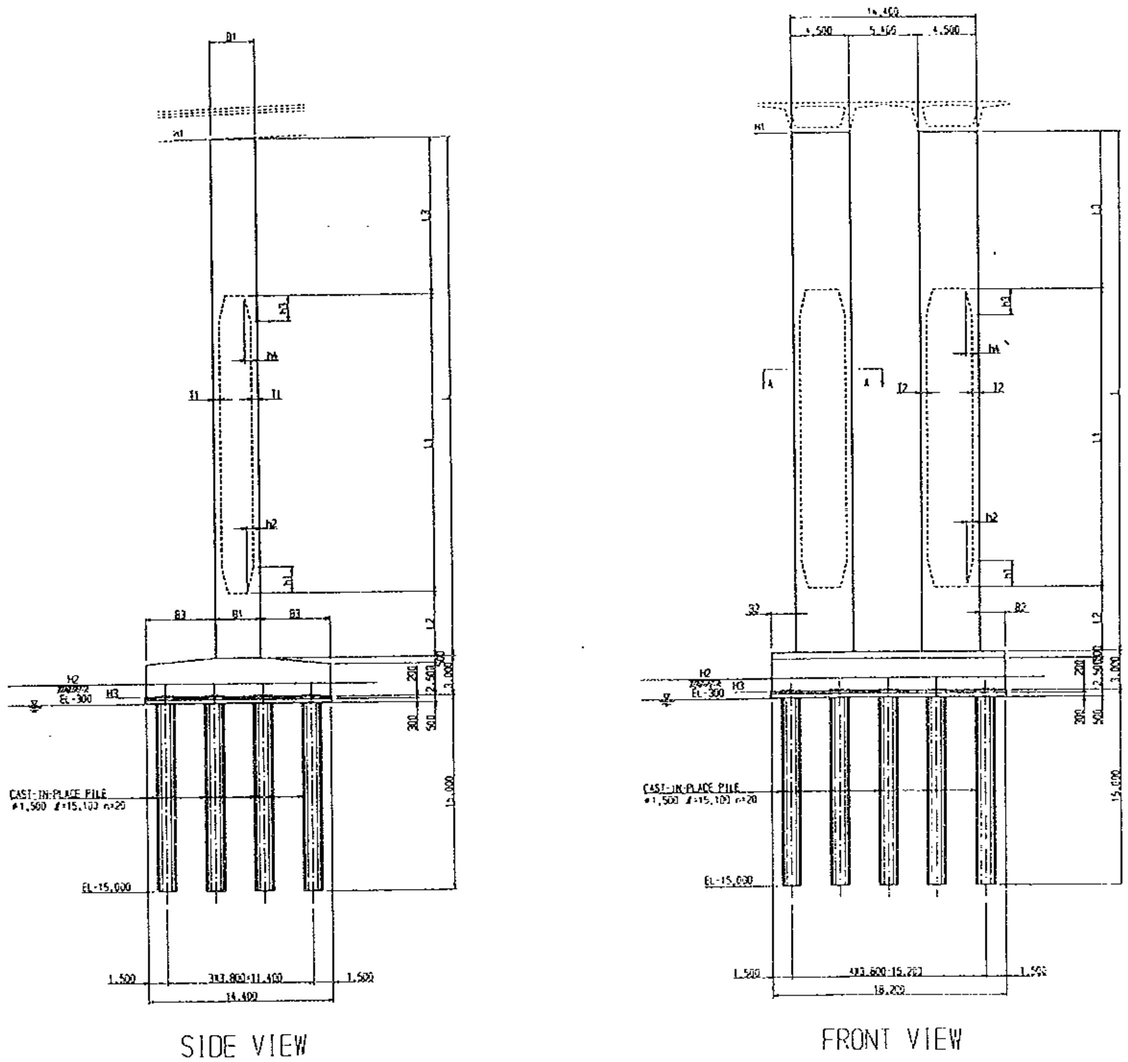


Fig. 4.2.4 General View of Substructure for Approach Bridges

Table 4.2.3 List of Substructures on the West Bank

Pier No.	Bridge Type	Span Length (m)	Planning Height (m)	Pier Height (m)	Pier Type	Full Part Top	Hollow Part	Full Part Bottom
AW			10.552	4.911	Full Section			
PW 1		40.000	11.872	5.510	Full Section			
PW 2		40.000	13.192	6.830	Full Section			
PW 3	BRW1 GIRDER	40.000	14.512	8.151	Full Section			
PW 4		40.000	15.832	9.471	Full Section			
PW 5		40.000	17.152	10.791	Full Section			
PW 6		30.500	18.159	11.797	Full Section			
PW 7	BRW2 GIRDER	40.000	19.479	13.117	Full Section			
PW 8		40.000	20.799	14.437	Full Section			
PW 9		30.500	21.805	15.443	Full Section			
PW 10		38.800	23.085	16.724	Full Section			
PW 11		38.800	24.366	18.004	Full Section			
PW 12	BRW3 GIRDER	38.800	25.646	19.285	Full Section			
PW 13		38.800	26.927	20.565	Full Section			
PW 14		38.800	28.207	21.846	Full Section			
PW 15		30.500	29.214	22.852	Full Section			
PW 16	BRW4 GIRDER	40.000	30.534	24.172	Full Section			
PW 17		40.000	31.854	25.492	Full Section			
PW 18		30.500	32.860	26.499	Hollow	12.0	9.5	5.0
PW 19		37.250	34.089	27.728	Hollow	12.0	10.7	5.0
PW 20		37.250	35.319	28.957	Hollow	12.0	12.0	5.0
PW 21	BRW5 RIGID FRAME	37.250	36.548	30.186	Hollow	12.0	13.2	5.0
PW 22		37.250	37.777	31.416	Hollow	12.0	14.4	5.0
PW 23		37.250	39.006	32.645	Hollow	12.0	15.6	5.0
PW 24		37.250	40.236	33.874	Hollow	12.0	16.9	5.0
PW 25		40.000	41.556	35.194	Hollow	12.0	18.2	5.0
PW 26		40.000	42.876	36.514	Hollow	12.0	19.5	5.0
PW 27		40.000	44.196	37.834	Hollow	12.0	20.8	5.0
PW 28	BRW6 RIGID FRAME	40.000	45.516	39.154	Hollow	12.0	22.2	5.0
PW 29		40.000	46.836	40.474	Hollow	12.0	23.5	5.0
PW 30		40.000	48.156	41.794	Hollow	12.0	24.8	5.0
PW 31		40.000	49.476	43.114	Hollow	12.0	26.1	5.0
PW 32		40.000	50.796	44.434	Hollow	12.0	27.4	5.0
PW 33		40.000	52.116	45.754	Hollow	12.0	28.8	5.0
PW 34		40.000	53.436	47.074	Hollow	12.0	30.1	5.0
PW 35	BRW7 RIGID FRAME	40.000	54.756	48.394	Hollow	12.0	31.4	5.0
PW 36		40.000	56.076	49.714	Hollow	12.0	32.7	5.0
PW 37		40.000	57.396	51.034	Hollow	12.0	34.0	5.0
PW 38		40.000	58.716	52.354	Hollow	12.0	35.4	5.0
PW 39		40.000	60.036	53.674	Hollow	12.0	36.7	5.0
PW 40		40.000	61.356	54.994	Hollow	12.0	38.0	5.0
PW 41		40.000	62.676	56.314	Hollow	12.0	39.3	5.0
PW 42	BRW8 RIGID FRAME	40.000	63.996	57.634	Hollow	12.0	40.6	5.0
PW 43		40.000	65.316	58.954	Hollow	12.0	42.0	5.0
PW 44		40.000	66.636	60.274	Hollow	12.0	43.3	5.0
PW 45		40.000	67.956	61.594	Hollow			
	Main Bridge (Cable-stayed Bridge)							

Table 4.2.4 List of Substructures on the East Bank

Pier No.	Bridge Type	Span Length (m)	Planning Height (m)	Pier Height (m)	Pier Type	Full Part Top	Hollow Part	Full Part Bottom
AE	BRE1 RIGID FRAME		20.435		Full Section			
PE 1		40.000	21.755	15.394	Full Section			
PE 2		40.000	23.075	16.714	Full Section			
PE 3		40.000	24.395	18.034	Full Section			
PE 4		40.000	25.715	19.354	Full Section			
PE 5	BRE2 RIGID FRAME	40.000	27.035	20.674	Full Section			
PE 6		40.000	28.355	21.994	Full Section			
PE 7		40.000	29.675	23.314	Full Section			
PE 8		40.000	30.995	24.634	Full Section			
PE 9		40.000	32.315	25.954	Full Section			
PE 10	BRE3 RIGID FRAME	40.000	33.635	27.274	Full Section	12.000	10.3	5.000
PE 11		40.000	34.955	28.594	Full Section	12.000	11.6	5.000
PE 12		40.000	36.275	29.914	Full Section	12.000	12.9	5.000
PE 13		40.000	37.595	31.234	Full Section	12.000	14.2	5.000
PE 14		40.000	38.915	32.554	Full Section	12.000	15.6	5.000
PE 15	BRE4 RIGID FRAME	40.000	40.235	33.874	Full Section	12.000	16.9	5.000
PE 16		40.000	41.555	35.194	Full Section	12.000	18.2	5.000
PE 17		40.000	42.875	36.514	Full Section	12.000	19.5	5.000
PE 18		40.000	44.195	37.834	Hollow	12.0	20.8	5.0
PE 19		40.000	45.515	39.154	Hollow	12.0	22.2	5.0
PE 20		40.000	46.835	40.474	Hollow	12.0	23.5	5.0
PE 21		40.000	48.155	41.794	Hollow	12.0	24.8	5.0
PE 22	BRE5 RIGID FRAME	40.000	49.475	43.114	Hollow	12.0	26.1	5.0
PE 23		40.000	50.795	44.434	Hollow	12.0	27.4	5.0
PE 24		40.000	52.115	45.754	Hollow	12.0	28.8	5.0
PE 25		40.000	53.435	47.074	Hollow	12.0	30.1	5.0
PE 26		40.000	54.755	48.394	Hollow	12.0	31.4	5.0
PE 27		40.000	56.075	49.714	Hollow	12.0	32.7	5.0
PE 28		40.000	57.395	51.034	Hollow	12.0	34.0	5.0
PE 29	BRE6 RIGID FRAME	40.000	58.715	52.354	Hollow	12.0	35.4	5.0
PE 30		40.000	60.035	53.674	Hollow	12.0	36.7	5.0
PE 31		40.000	61.355	54.994	Hollow	12.0	38.0	5.0
PE 32		40.000	62.675	56.314	Hollow	12.0	39.3	5.0
PE 33		40.000	63.995	57.634	Hollow	12.0	40.6	5.0
PE 34		40.000	65.315	58.954	Hollow	12.0	42.0	5.0
PE 35		40.000	66.635	60.274	Hollow	12.0	43.3	5.0
PE 36		40.000	67.955	61.594	Hollow			
	Main Bridge (Cable-stayed Bridge)							

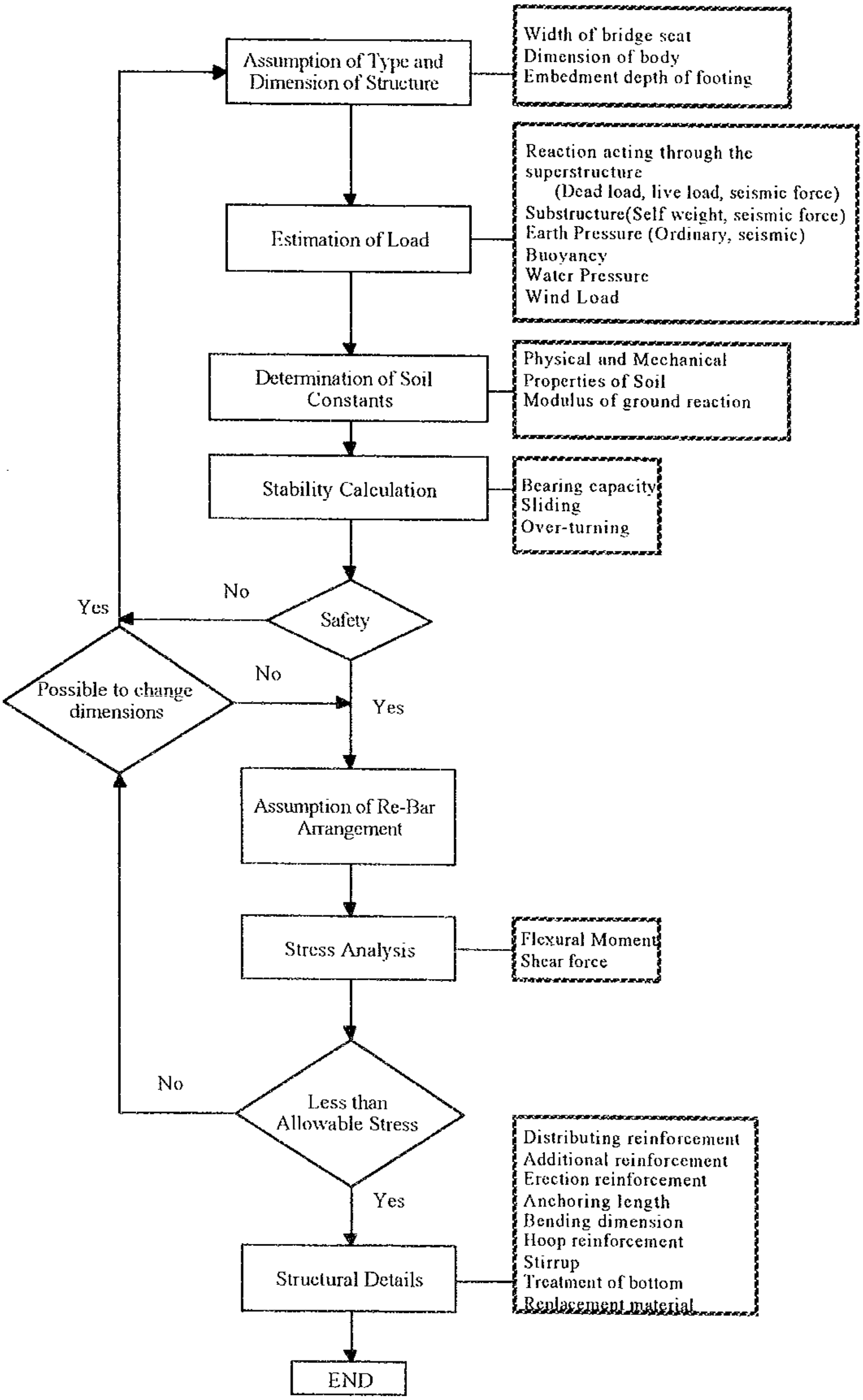


Fig. 4.2.5 General Procedure for Design of Substructure

4.2.4 Superstructures

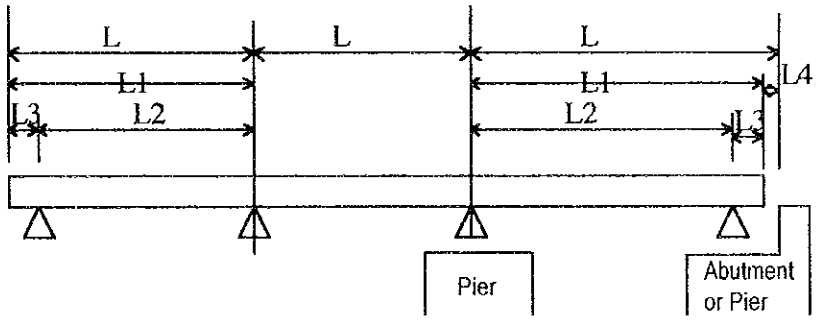
(1) General Conditions

Bridge type	:	Mullet-span continuous girder Mullet-span continuous rigid frame
Girder type	:	Post-tensioned prestressed concrete box girder
Bridge width	:	10.050m 9.730m
Carriageway width	:	7.300m (2@3.650m)
Live load	:	Live load to Egyptian codes

The typical cross sections of girder are shown in Figure 4.2.6.

(2) Structural Details

Required details of bridge length and span arrangement are shown below:



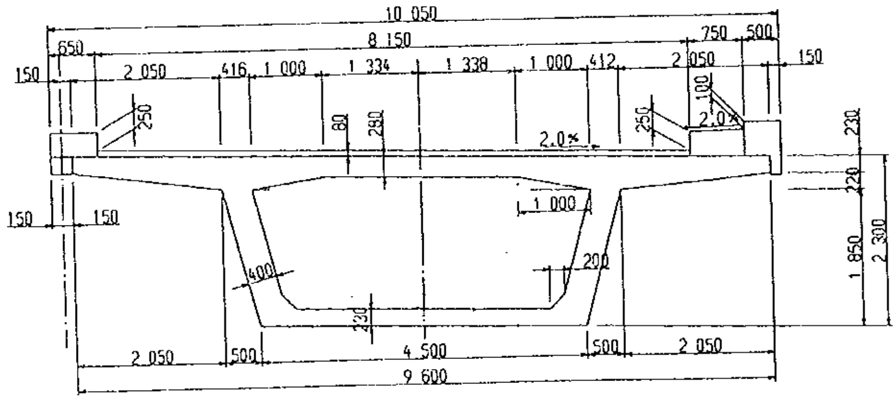
Where:

L	:	Bridge span length
L1	:	Girder length
L2	:	Girder span length
L3	:	Girder edge overhang length
L4	:	Movement Gap

The following lengths have been selected for each superstructure.

Table 4.2.5 Bridge Length Detail

L	L1	L2	L3	L4	Application
40.000m	39.930m	39.400m	530mm	70mm	BRW1, BRW6～BRW8 BRE1～BRE6
38.800m	38.730m	38.200m	530mm	70mm	BRW3
34.050m	33.980m	33.450m	530mm	70mm	BRW2, BRW4
26.350m	26.280m	25.750m	530mm	70mm	BRW2, BRW4
37.250m	37.180m	36.650m	530mm	70mm	BRW5



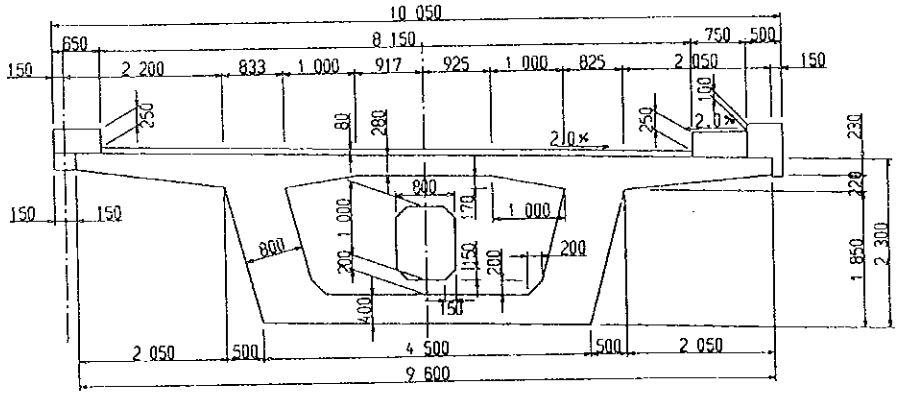


Fig. 4.2.6 Typical Cross Sections of Girder

(3) Calculation Method

(a) Procedure for Design Work

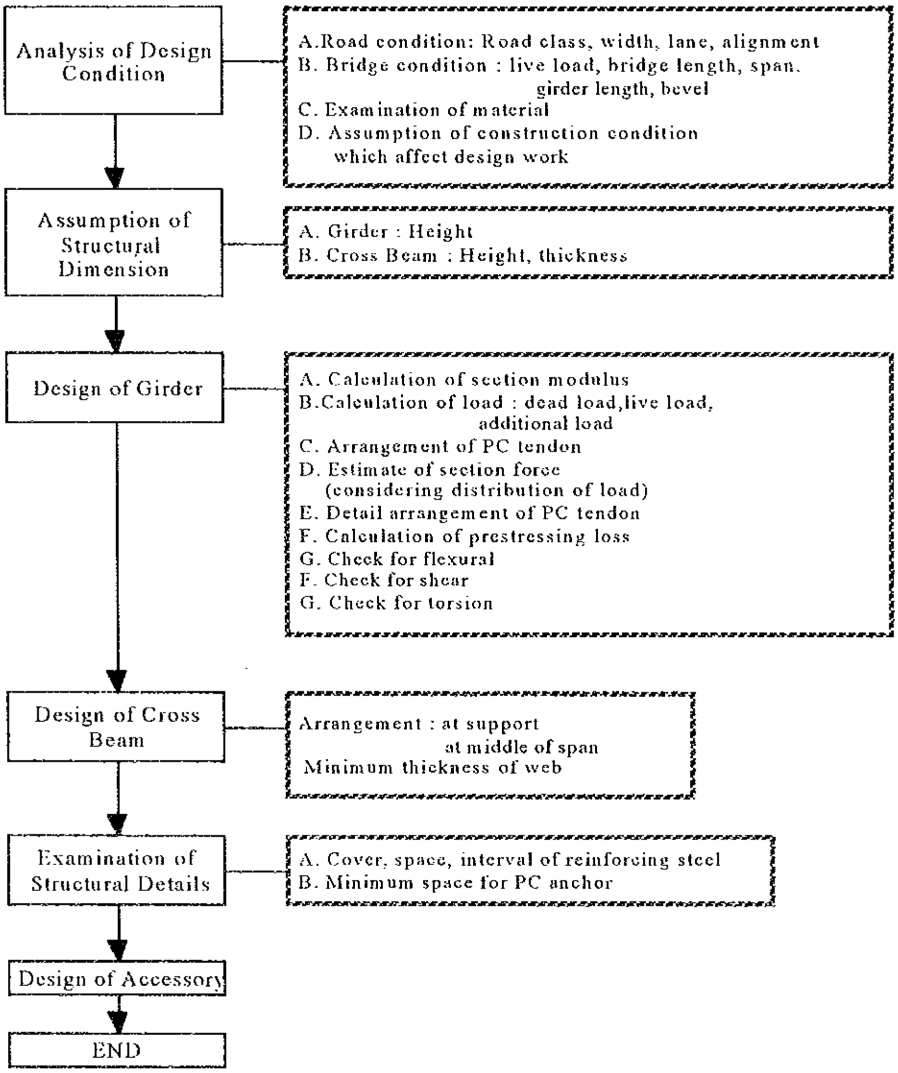


Fig. 4.2.7 General Procedure for Design of Superstructure

(b) Structural Analysis

There are two types of superstructure for the approach bridges, but both types of superstructures have the same configuration which is a single box. Therefore, the superstructure is considered as a beam stricture in the analysis model and the sectional forces of superstructure have been calculated using the beam theory.

The two dimensional frame model to show the entire bridge has been used for the structural analysis.

(c) Calculation of Effective Prestress

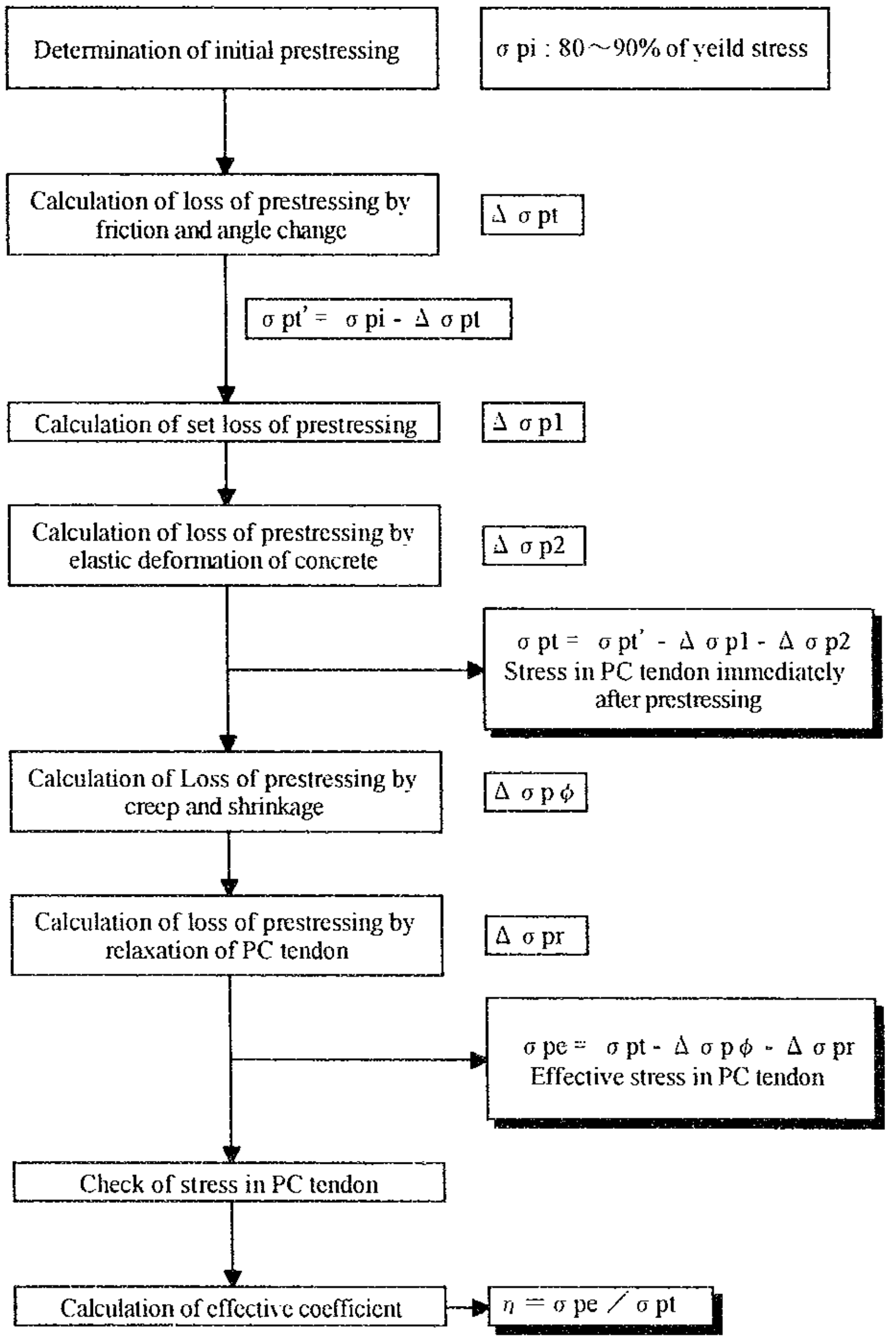


Fig. 4.2.8 General Procedure for Calculation of Effective Prestress

(d) Examination of safety against collapse

Prestressed concrete members have been examined for both cases of design loading and ultimate loading.

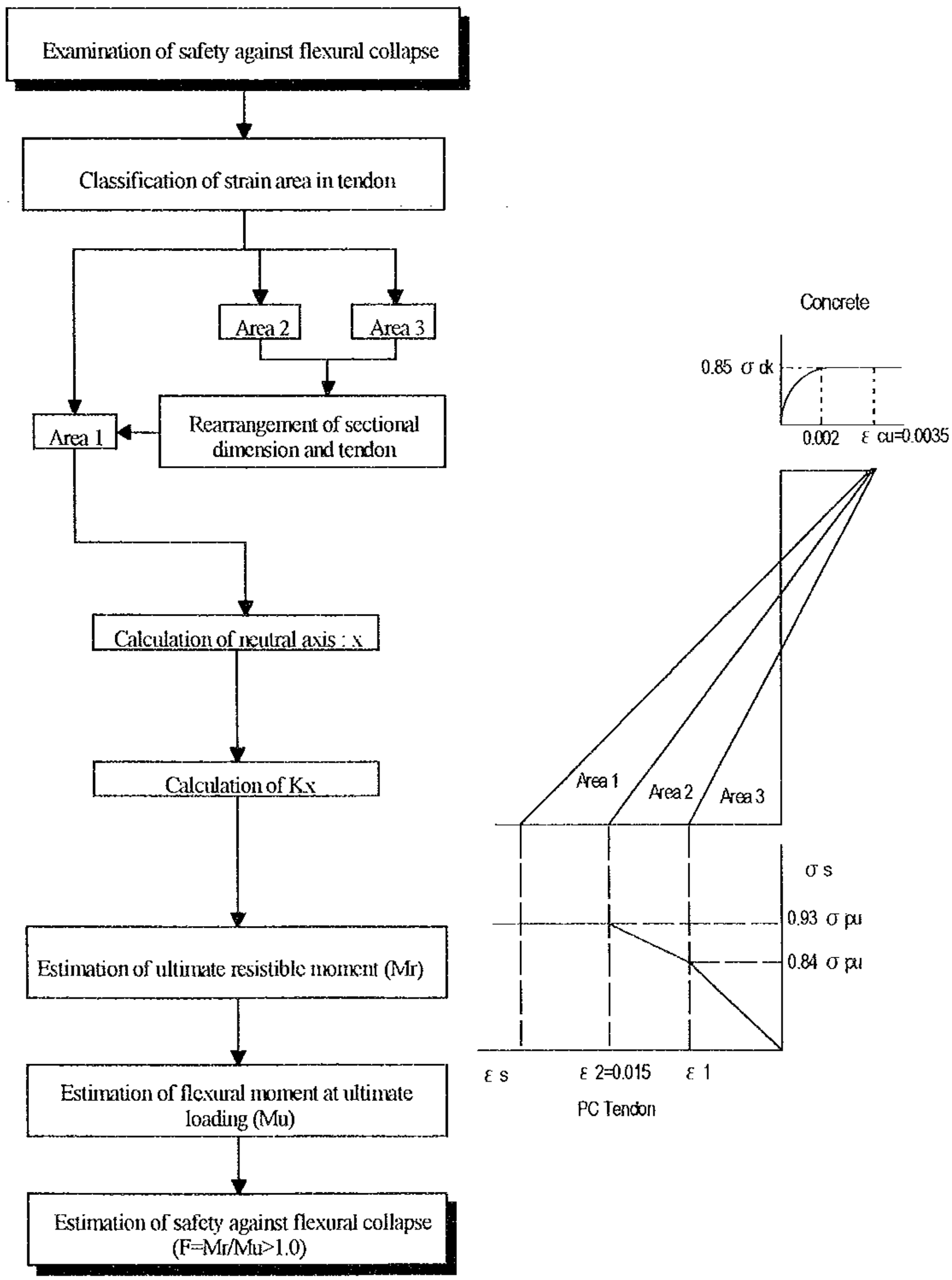


Fig. 4.2.9 General Flow for Examination of Ultimate Loading

4.3 Calculation Analysis for Approach Roads and Access Roads

The geometric design of the road crossing the Suez Canal at Qantara has been carried out based on the geometric design criteria shown in Chapter 2.

The following items have been studied in this paragraph.

- Horizontal and Vertical Alignment (Plan and Profile)
- Embankment Stability
- Pavement Design
- Slope Protection
- Drainage
- Lighting System
- Traffic Safety Facilities (Traffic Barrier)
- Traffic Management Facilities
- Miscellaneous

4.3.1 Horizontal and Vertical Alignment

(1) Horizontal Alignment

1) Connecting Roads

In principle, the road crossing the Canal will connect directly into the Cairo - Ismailiya - Port Said Road on the West Bank and the New Central Road on the East Bank.

a. The West Bank

The alternative methods of connecting the Canal road crossing into the Cairo - Ismailiya - Port Said Road on the West Bank have been studied. In addition, the possibility of connecting into the Abou Souwer - Qantara Road on the West Bank has been also examined as one of the alternatives.

As a result of this study, the Canal road crossing will be connected into the Cairo - Ismailiya - Port Said Road 300m north of the intersection of the Cairo - Ismailiya - Port Said Road with the Abou Souwer - Qantara Road on the West Bank.

b. The East Bank

The New Central Road is the only existing main road close to the Suez Canal on the East Bank. Therefore, the Canal road crossing will have to connect directly into the New Central Road to provide the preferred horizontal alignment.

2) Crossing Location

There are some local constraints in the area which will limit the selection of the most appropriate route for the Canal road crossing. It will not be possible to avoid all of these constraints. However, the route has been selected to avoid passing over the most sensitive institutions such as military areas and public facilities including, the schools and irrigation canal office as far as possible. In addition, minimizing the demolition of private houses in the village to the west of the irrigation canal was taking into account in determining the crossing location.

The following three alternatives for crossing locations have been selected and compared in order to determine the best crossing alignment.

- Alternative 1 : SCA km 48 + 450
- Alternative 2 : SCA km 48 + 517.5
- Alternative 3 : SCA km 48 + 505

As a result of this study, SCA km 48 + 505 is considered to be the best crossing location for the following reasons.

- To avoid intruding into the military area and demolishing private houses are the most important factors to be considered.
- The military area where the road will cross is small and relatively unimportant. The number of demolished houses will be small, and the removal or interference with public institutions is less sensitive than that of private houses.
- The span arrangement of the approach viaduct crossing over the Cairo - Ismailiya - Port said Road is simplified.

3) Factors Dictating Horizontal Alignment

The horizontal alignment has been determined by the following factors.

a. The road crossing the Canal is to connect at the straight sections of the New Central Road and the Cairo - Ismailiya - Port Said Road.

b. The crossing road is to be straight on the bridge section.

c. The radius of curves for the road crossing are to be as large as possible. If possible, a radius of 2,000m or more which will not require a transition curve will be provided.

d. The road must cross over the Suez Canal at right angles and at a point where the alignment of the Suez Canal is straight.

e. The following very important institutions are to be avoided by the road crossing as far as possible;

- On the West Bank : Military Area, Irrigation Canal Office, Schools
- On the East Bank : Fish Feeding Area

4) Horizontal Alignment

The horizontal alignment of the Canal road crossing has been studied and selected as shown on the plan in Paragraph (3).

(2) Vertical Alignment

1) Critical Height

a. Height of Crest

The clearance provided between the proposed elevation of the road and the outer limit of the navigation clearance envelope of the Suez Canal is about 2.9 m taking into consideration the girder depth and the deflection of the main bridge. The elevation of the crest of the road has been decided by the above clearance and the vertical grade. (Refer to Fig. 4.3.1)

The proposed elevation at the outer limit of the navigation clearance envelope
PHo = 73.652 m
The proposed height of the crest PHc = 79.988 m → PHc = 80.000 m

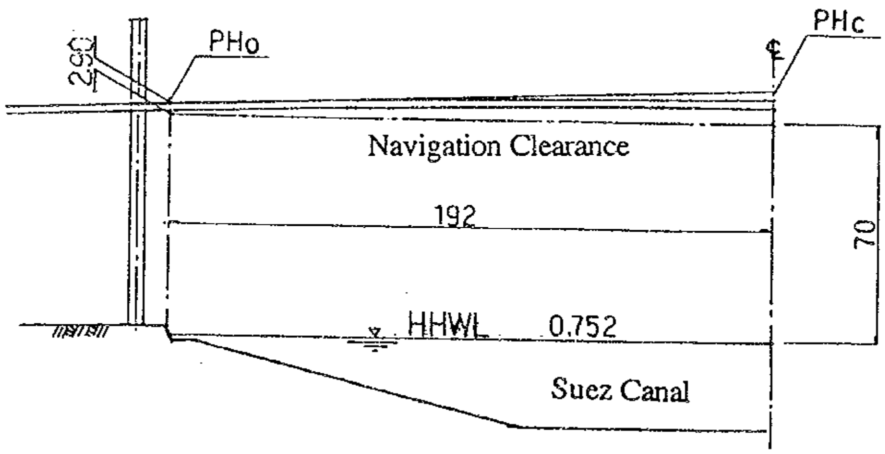


Fig. 4.3.1 Relationship between Proposed Elevation and Navigation Clearance

b. Height of Access Road

The heights of the access road embankments are 2.0m to 3.0m above the ground level taking account of crossing structures for pedestrians and irrigation.

Therefore the proposed elevations of the access roads are 4.0m on the West Bank and 2.5m on the East Bank.

2) Vertical Grade

A constant vertical grade has been adopted for both sides of the elevated sections of the Canal road crossing and vertical grades of 3.3 % and 4.0 % have been studied in order to determine the appropriate grade of the Canal road crossing.

As a result of this study, a vertical grade of 3.3% has been selected because of the resultant smoother traffic flows taking account of the characteristics of Egyptian vehicles. In addition when considering the heavy vehicle ratio about 1 in 5 on the road crossing, and the environmental issues of the road, including noise and air pollution, a vertical grade of 3.3% is preferable.

(3) Plan and Profile

The plan and profiles of the road crossing the Suez Canal have been determined based on the studies in Paragraphs (1) and (2), and the proposed plan and profiles are shown in Fig. 4.3.2 and Fig. 4.3.3.

4.3.2 Embankment Stability

(1) Maximum Height

The maximum height of the approach embankments has been determined by the two key factors of stability and the site conditions along the approach embankment alignment on each side of the Canal.

1) Stability of Embankment

The two conditions controlling stability of embankments which have been studied are slope failure and bearing capacity of the ground.

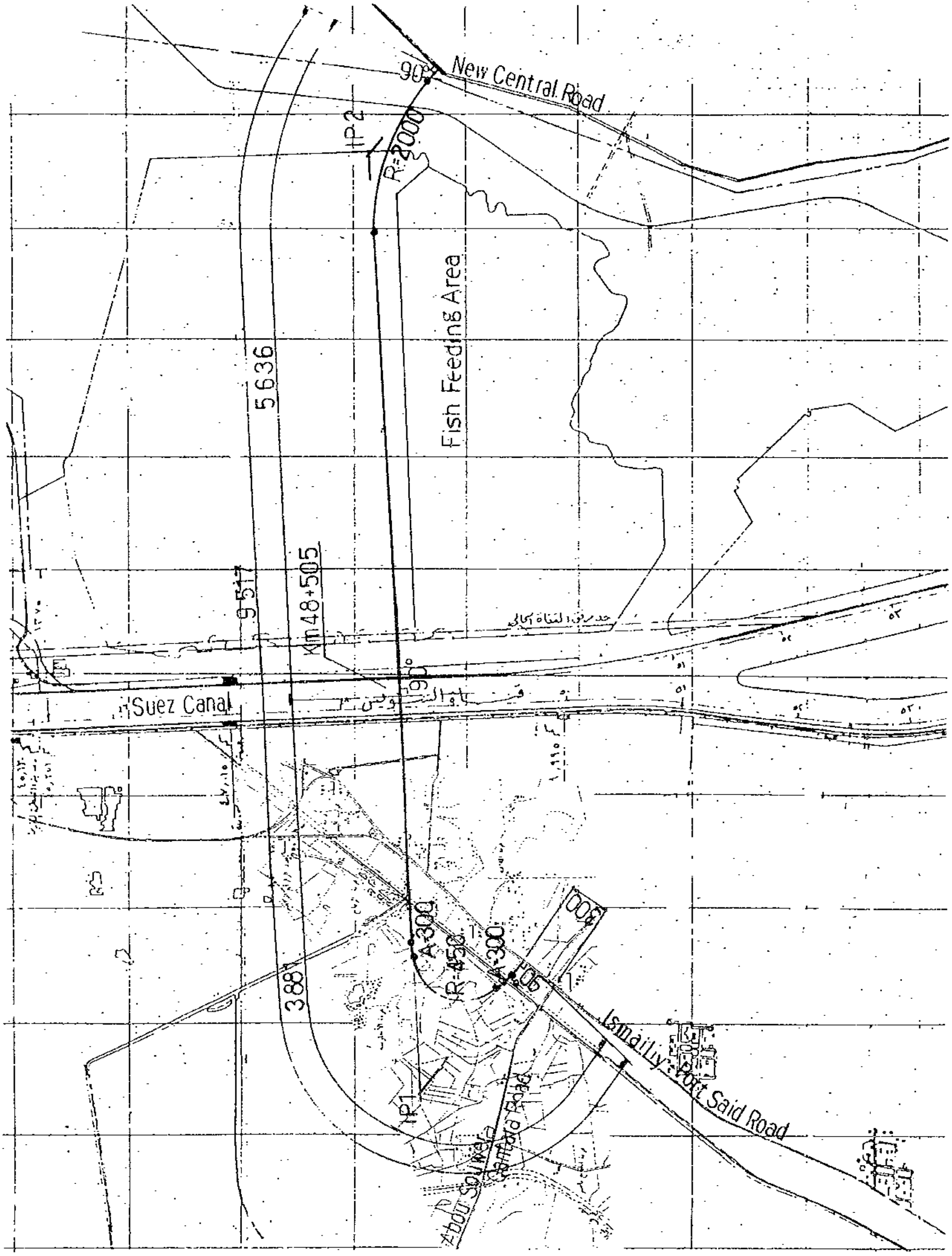


Fig. 4.3.2 Plan of the Canal Road Crossing

THE DETAILED DESIGN STUDY ON THE PROJECT FOR CONSTRUCTION OF THE SUEZ CANAL BRIDGE

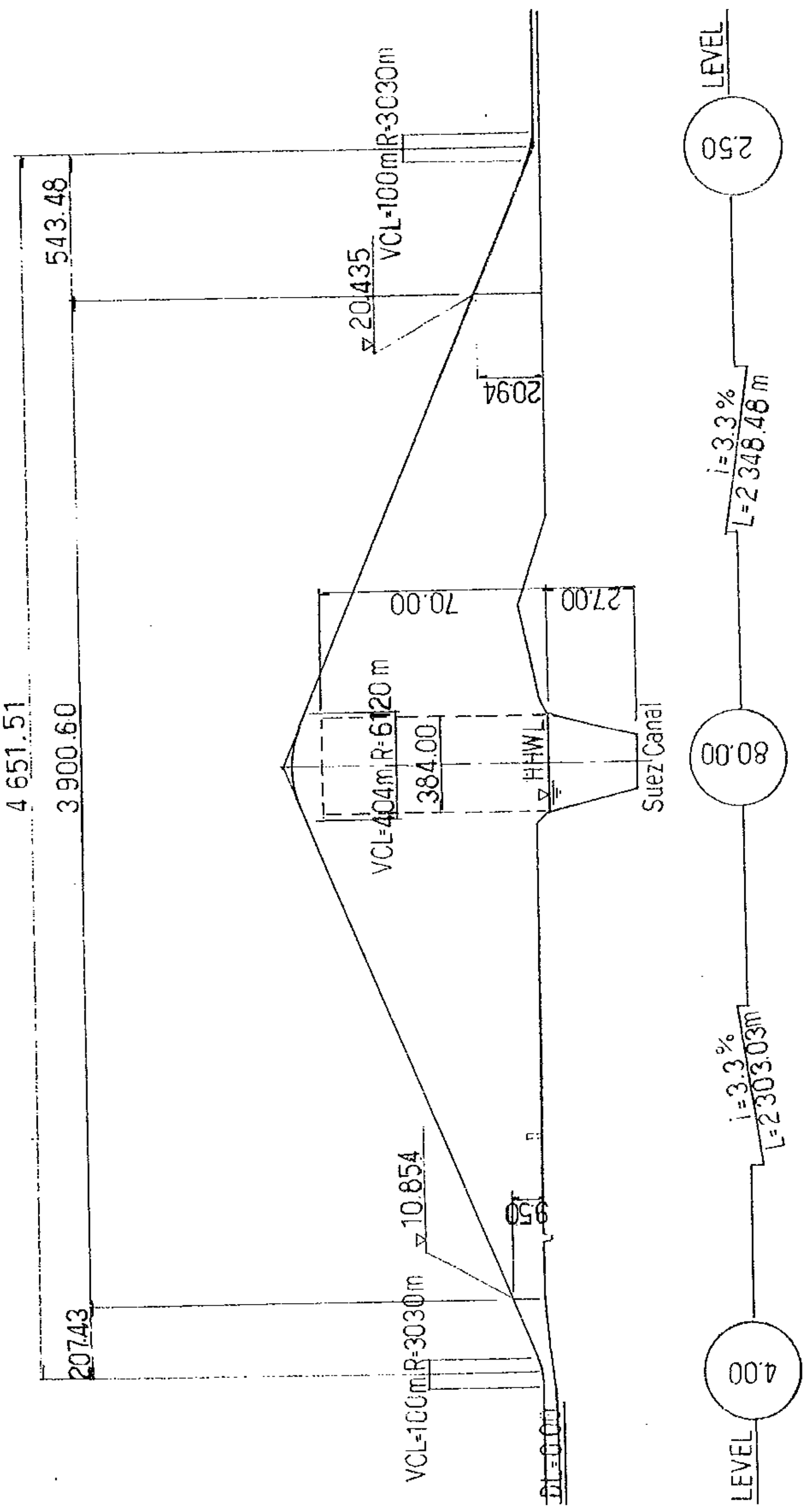


Fig. 4.3.3 **Vertical Grade of the Canal Road Crossing**

THE DETAILED DESIGN STUDY ON THE PROJECT FOR CONSTRUCTION OF THE SUEZ CANAL BRIDGE

a. Slope Failure

Safety factors for slope failure with embankment heights of 10m on the West Bank and 20m on the East Bank were calculated. The result of these calculations are as follows;

- Assuming the internal angle of friction of the embankment is 35 degrees, the safety factor for shallow surface failures is greater than 1.2, and
- The safety factor of failures reaching to the ground is greater than 1.5.

The allowable safety factor is 1.2. Thus, the approach embankment heights of 10m on the West Bank and 20m on the East Bank are stable against slope failures.

Table 4.3.1 Results of Stability Calculations of Slope Failures

Location	Height H (m)	Slope Inclination n : 1	IFA ϕ (degree)	Safety Factor	
				Embankment Fs_1	Foundation Fs_2
West Bank (Main Land)	10.0	2 : 1	35	1.21	2.07
East Bank (Sinai Side)	20.0	2 :1	35	1.28	1.52

Source : Study Team

IFA : Internal Friction Angle of Embankment

Allowable Safety Factor : Fsa = 1.2

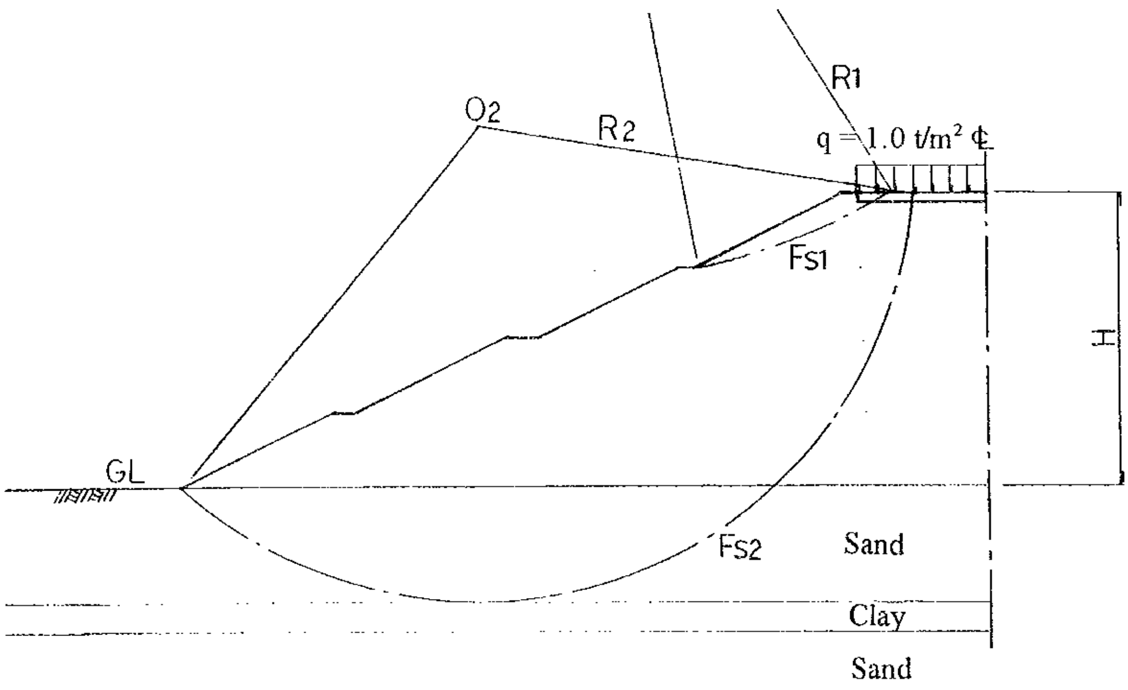


Fig. 4.3.4 Results of Stability Calculations of Slope Failures

b. Bearing Capacity

The bearing capacities required for 10m and 20m height embankments are 19 t/m^2 and 38 t/m^2 respectively. The actual bearing capacities of the ground have been estimated to be 20 t/m^2 on the West Bank and 40 t/m^2 on the East bank based upon the data of the geotechnical investigations. Therefore, the approach embankments of these heights are considered to be stable.

c. Maximum Height for Embankment Stability

As a result of this study, the maximum heights of the approach embankments will be 10m on the West Bank and 20m on the East Bank.

2) Site Conditions

a. The West Bank

There is an irrigation canal, schools and private houses near the proposed location of the bridge abutment. In order to minimize the demolition of these institutions and buildings, the abutment should be situated to the west of the school or village to the west of the irrigation canal. This will reduce the height of the approach embankment to less than 10m on the West Bank.

b. The East Bank

There are no facilities which will influence the height of the approach embankment on the East Bank.

3) Maximum Height

As a result of the study, the maximum heights of the approach embankments will be less than 10m on the West Bank and about 20m on the East Bank.

(2) Material and Structure

1) Material

In order to ensure the stability of the approach embankments, an internal angle of friction of 35 degrees is required in the shallow surface sections of the embankments.

Therefore, unscreened gravel and gravely soil or fine sand from a quarry for the critical sections of the embankment, such as the toes and surfaces of slopes, and sand around the site will be used for the other parts of the embankment.

The materials of the approach embankment will be fully selected after further studies in the detailed design.

2) Structure

In order to ensure the stability of the approach embankments, the structure of the embankments will be further studied in the detailed design. An example of an embankment structure is shown in Fig. 4.3.5.

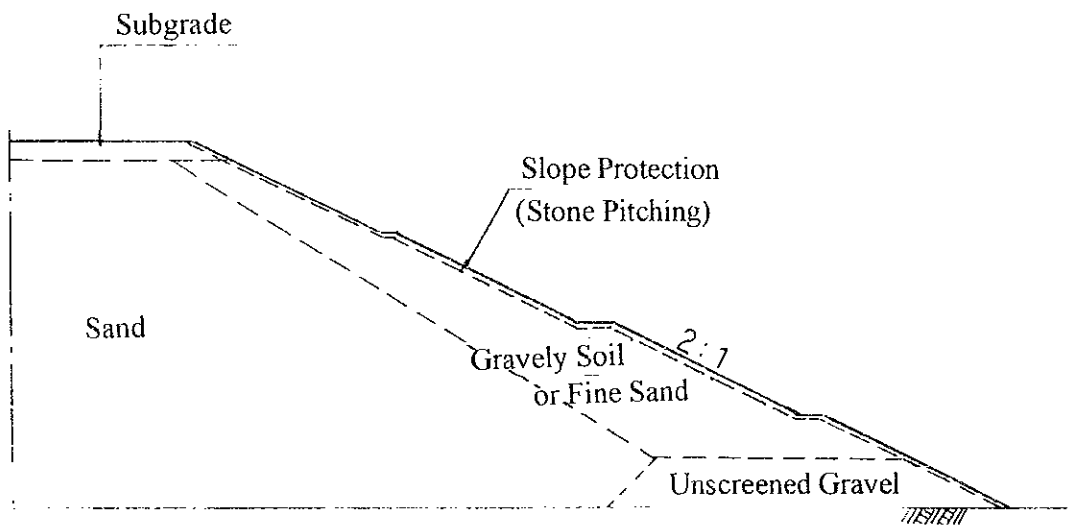


Fig. 4.3.5 Structure of Approach Embankments

(3) Side Slopes

As a result of the embankment stability calculations, the inclination of the approach embankment slopes should not be steeper than 2:1 (horizontal : vertical).

The inclination of 2:1 will also be suitable for the slopes of the access road which have a height of between 2.0m and 3.0m, based on the Geometric Design Standards in Egypt.

(4) Slope Protection

Slope protection for preventing the surface from erosion and weathering is necessary.

Stone pitching will be required to protect the surface of the embankments after taking into consideration the local climate and embankment material.

4.3.3 Pavement Design

(1) General Description

Asphaltic concrete pavement which is commonly used, will be provided for the two top courses of the pavement on the approach embankments and access roads.

The standard pavement structure in Egypt will be used for the Canal road crossing. The thickness of pavement has been reviewed using the American standard (AASHTO) from the results of the geotechnical investigation. The thickness designed will be reviewed based on the actual CBR from the field tests during the construction.

(2) Review of Pavement Thickness

The standard thickness of pavement in Egypt is as shown in Fig. 4.3.6.

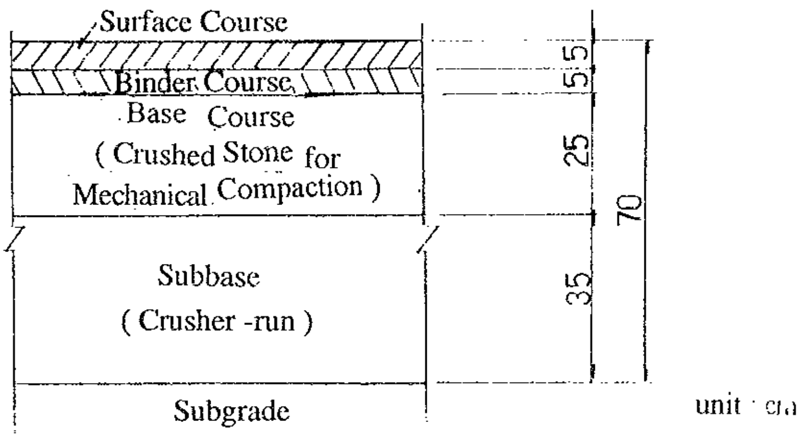


Fig. 4.3.6 Structure of Pavement

(3) Design Thickness

The thickness of pavement has been reviewed using the AASHTO standard. A result of the review is shown below, and the thickness decided using the Egyptian standard complies with the AASHTO standard.

The thickness of pavement is calculated taking into consideration axle load of vehicle, frequency of vehicles, and stiffness of subgrade in AASHTO.

1) Design Criteria

- Axle Load of Vehicle	:	10 t
- Frequency of Vehicles	:	3,000 vehicles/hr
- Base Course	:	CBR = 8

2) Pavement Thickness by AASHTO

As the result of the Calculation based on AASHTO, the thickness of pavement should be 45 cm. The thickness of each part is as follows;

- Surface Course	:	5	cm
- Binder Course	:	5	cm
- Base Course	:	25	cm
- Subbase	:	25	cm
- Total	:	60	cm

The thickness of pavement decided using the Egyptian standard thickness complies with the thickness of pavement calculated by the AASHTO standard. Therefore, the thickness of 60 cm will be used for the pavement in the earthwork sections of the Suez Canal road.

4.3.4 Road Facilities

(1) Drainage

Taking account of the climate in Egypt, full drainage facilities are not required because the precipitation at the site is very low.
Therefore, toe drains will not be provided for embankments. Water on the road surface will be drained via the shoulders.

(2) Lighting System

A lighting system will be provided for traffic safety and to encourage smooth traffic flows at night. Lighting poles will be installed on both sides of the road at 20m intervals on the approach embankments and access roads.

The necessary level of lighting to maintain safe and smooth traffic flows, and the type and structure of lighting poles have been fully studied and selected.

(3) Traffic Safety Facilities

1) General Description

In order to maintain safe traffic operation and to minimize traffic accidents, traffic safety facilities such as curbstones, traffic islands, traffic barriers and pedestrian subways should be provided for the new road crossing.

2) Traffic Barrier

The strength of the traffic barrier has been checked by BS standard. A result of the calculation to ensure the strength of the traffic barrier provided for the approach road is shown below.

As a result of the calculation, the traffic barrier is confirmed to be strong enough against collision by vehicles.

(4) Traffic Management Facilities

Traffic management facilities including traffic signs, road markings, emergency telephones, road information boards and observation facilities will be provided to encourage smooth traffic flows and to reduce traffic accidents. As roundabout type intersections will be provided at the intersections between the new road crossing and the Cairo - Ismailiya - Port Said Road and the New Central Road, traffic lights are not provided at these intersections.

Traffic sign and indication boards are provided taking into account the long and steady vertical grade of the approach bridge and roads.

4.3.5 Bridge over Abassah Canal

(1) Dimensions of Bridge

This bridge is located 200 m north west of the intersection between the road crossing the Canal and the Cairo - Ismailiya - Port Said road on the access road on the West bank. The dimensions of the bridge over Abassah Canal are as follows.

- Bridge Type	:	PC Simple T-Girder
- Length	:	20 m
- Width	:	9.80 m × 2 = 19.80 m
- Abutment	:	RC Reversed T Type
- Foundation	:	Spread Foundation

(2) Design Criteria and Design method

The design criteria and design method which are used for the design of the approach bridges are provided for the design of this bridge.

(3) Result of Calculation

The results of the calculation are shown below.

- Girder Height 1.35 m
- Foundation 12.05 m×2.70 m

4.3.6 Crossing Structures

(1) General Description

In order to maintain waterways for cultivation and access routes for the local people around the new road, pipe culverts for irrigation waterways and box culverts for small vehicles and pedestrians will be provided in the approach embankments and access roads on the West Bank.

(2) Dimensions of Box Culverts

Three box culverts will be provided in the access road on the West bank. The dimensions of the box culverts are as follows.

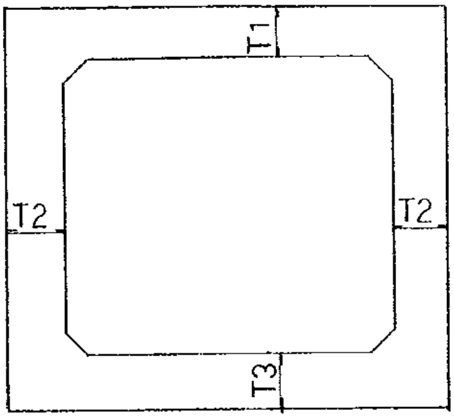
Descriptions	Unit	Box A	Box B	Box C
Item	-	Small Vehicle	Pedestrian	Light Vehicle
Inner Width	m	7.00	3.40	6.50
Inner Height	m	3.00	3.50	3.50
Length	m	25.0	25.0	25.0
Water Way	-	Without	With	With

(3) Design Criteria and Design method

The design criteria and design method which are used for the design of the approach bridges are provided for the design of this bridge.

(4) Result of Calculation

The results of the calculation are shown below.



Unit : cm

	Box A	Box B	Box C
T1	60	40	60
T2	40	40	40
T3	50	40	50

Fig. 4.3.7 Section of Box Culvert

4.3.7 Intersection Planning

(1) General Description

The intersections between the new road and the Cairo - Ismailiya - Port Said Road on the West Bank, and the New Central Road on the East Bank have been fully studied to provide smooth and safe traffic flows.

1) Design Vehicle : WB - 50

2) Design Speed

- The Suez Canal road : 80 km/hr
- The Cairo - Ismailiya - Port Said road : 100 km/hr
- The New Central road : 80 km/hr

(2) The West Bank

The intersection between the road crossing the Canal and the Cairo - Ismailiya - Port Said road has been studied and types of this intersection have been determined taking into consideration the traffic demand and construction costs.

1) Intersection Type at the Opening Year

The number of vehicles which will use this road crossing the Canal is expected to be less than 10,000 a day in 2002. Therefore, a grade crossing with a semi-roundabout will be provided for this intersection at commencement of service of the road crossing the Canal.

2) Intersection Type in the Target Year

The number of vehicles which will use this road crossing the Canal is expected to be 28,000 a day in 2017. Although a grade crossing is possible for this traffic volume, providing a grade separation should be considered following the increase of the traffic volume crossing the Canal. A flyover could be provided for the traffic through The Cairo - Ismailiya - Port Said road.

(3) The East Bank

The intersection between the road crossing the Canal and the New Central road has been studied and a roundabout type will be provided for this intersection.

4.4 Other Structures / Elements

4.4.1 Bridge Bearings for Main Bridge

There are five different types of bearings used in main bridge.
at Pylon:

a) rubber with steel sheet type, 500 ton capacity, vertially rigid, longitudinally spring (1000 ton/m) transversely free.

b) rubber pad lateral bearing, 650 ton capacity, transversely rigid, longitudinally free, vertically free.

at Auxiliary Piers (P1, P6)

c) pendal bearing, -370 ton capacity, vertically rigid, longitudinally free, transversely free.

d) metal wind bearing, 55 ton capacity, transversely rigid, longitudinally free, vertically free.

at Auxiliary Piers (P2-P5)

e) pendal bearing, -1100 ton capacity, vertically rigid, longitudinally free, vertically free.

f) metal wind bearing, 56 ton capacity, transversely rigid, longitudinally free, vertically free.

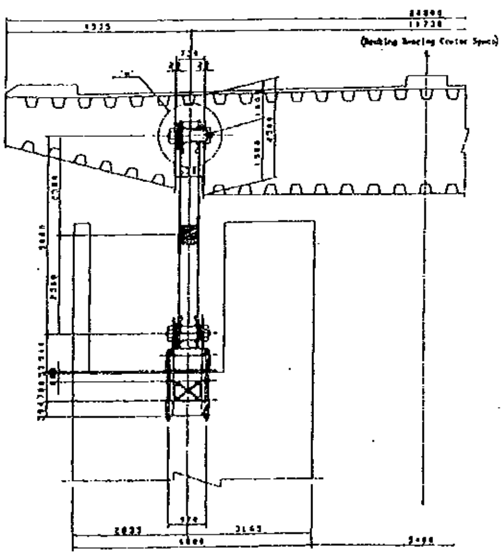


Figure Pendal Bearing

4.4.2 Expansion Joint for Main Bridge

As the expansion joint between the main girder end and the PSC girder end of the approach bridge, a expansion joint of steel finger type is used, due to large movement (700mm) of the deck girder under seismic condition. Long nose of the fingers are supported by slide bearings located at steel girder end. Concrete girder side is designed as equipped with spring bolts so as to absorb the inter angle between two fingers.

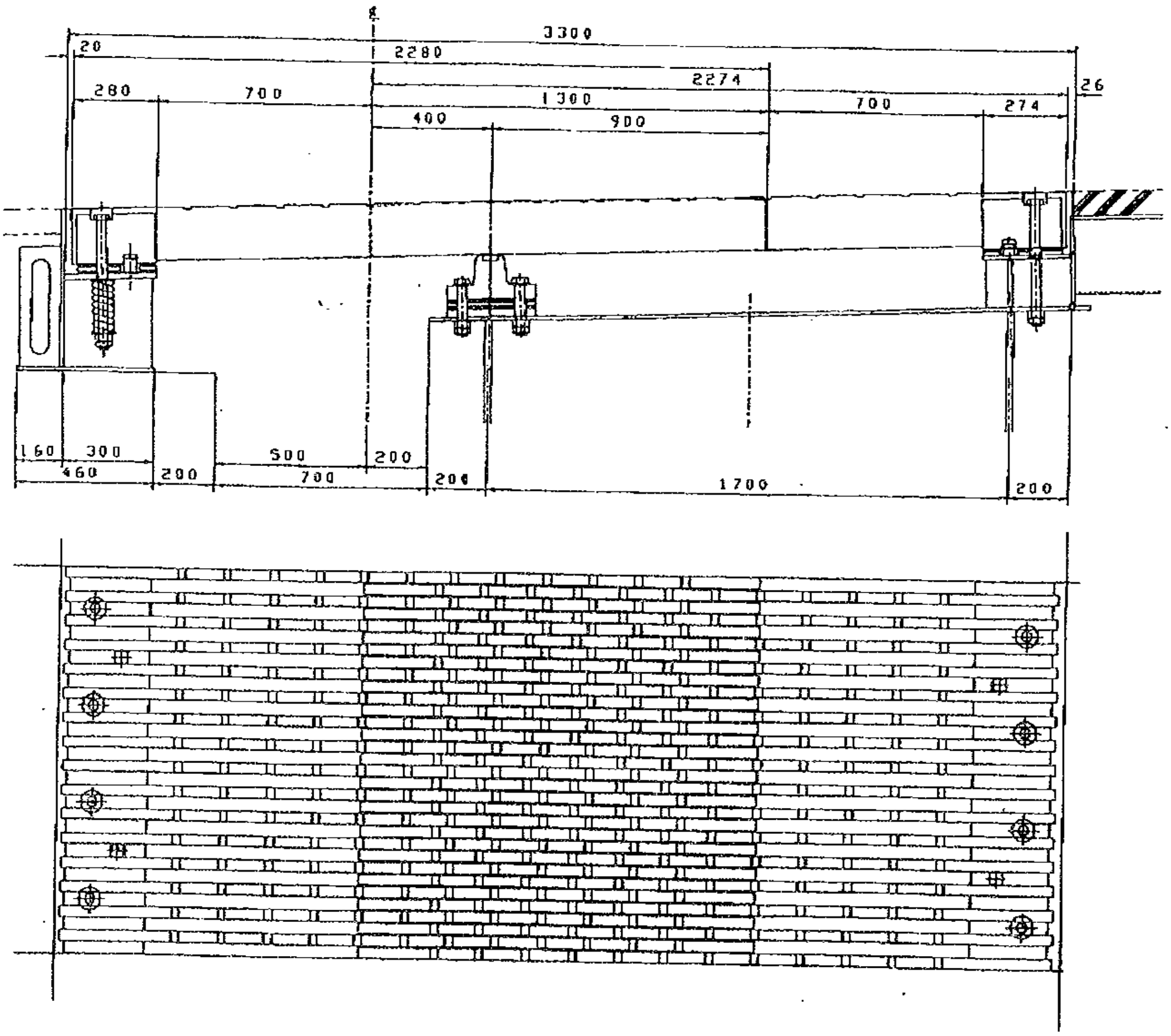


Figure Expansion Joint

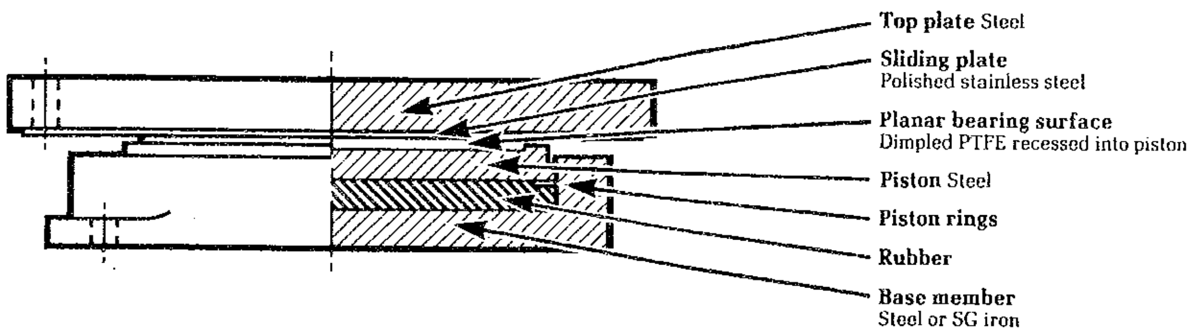
4.4.3 Bridge Bearings for Approach Bridges

(1) Type of Bridge Bearings

The type of bridge bearings for approach bridges was adopted "Pot Bearing". The pot bearing is mainly comprised with top plate, PTFE plate, piston steel, sealed rubber and base member. Free pot bearing corresponds to free support condition of piers and horizontal movement is to be absorbed by sliding between the top plate and PTFE plate and rotation is allowed by elastic deformation of rubber. Fixing pot bearing corresponds to fixed support condition. No sliding mechanism is required but rotation is absorbed by rubber deformation for the fixing bearing. Free or Fixing bearings shall be exactly installed corresponding to the support condition of piers.

(2) Typical component of pot bearing

Typical component of the pot bearing is shown as below;



4.4.4 Bridge Expansion Joint for Approach Bridges

(1) Type of Bridge Expansion Joint

Rubber expansion joint was adopted for approach bridges. Basic structure consists of a Neoprene rubber body reinforced with a steel bridge plate. Taking advantage of characters of both materials, high elasticity of rubber and stiffness of steel, rubber bridge expansion joint is designed to absorb and/or disperse all anticipated load-impact caused by the passing of vehicles on the expansion joint. Expansion and shrinkage of a girder results in movement of the expansion joint, which can be easily realized by shear deformation of rubber.

(2) Typical Component of Rubber Bridge Expansion Joint

Typical component of the rubber bridge expansion joint is shown as below;