ENGINEERING SERVICE
FOR
THUWANNA BRIDGE PROJECT
OF
BURMA

DESIGN REPORT

March 1980

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

The Japanese Government which has been extending technical cooperation in response to the request from the Government of the Socialist Republic of the Union of Burma to the "Bridge Engineering Training Centre in Burma" since last July decided to prepare a series of specifications relating to the THUWUNNA Bridge as teaching material at the Centre.

For this purpose, the Japan International Cooperation Agency (JICA), commissioned by the Japanese Government, dispatched to Burma a survey team headed by Mr. Kiyoshi Miyamoto, head of Structure & Engineering Division, Engineering Department, Japan Highway Public Corporation from August 19, 1979 for about two weeks. With the close cooperation of the Burmese authorities the team conducted its survey smoothly.

The team, after returning to Japan discussed the findings of the survey with the Burmese counterparts under training in Japan and prepared a draft report.

JICA sent another team to Burma in January 1980 to hold discussions based on that draft report with the Burmese officials and has compiled the present report.

I hope this report will prove to be useful for Burmese bridge engineers and contribute to the economic and social development of Burma as well as to the promotion of friendly relations between our two countries.

I wish to express my deep appreciation to the Government and officials concerned of the Union of Burma for their close cooperation extended to the survey team.

March, 1980

Keisuke Arita

Keisuke Arita
President
Japan International Cooperation Agency.

Introduction

Burma stretches longitudianlly from north to south (2,100km.), and 900km. from east to west. Three large rivers running from the north divide the country and thus impede communication between east and west. Bridges constructed across these rivers will improve transportation which is significant in the economic development of the country.

Burma produces its own cement although steel for the bridges is imported. Because of the availability of local resources, construction expenses are further lessened if Burmese are taught the construction techniques of long spanned, prestressed concrete bridges. To meet these needs, JICA invited specialists on the subject to the Thuwana Bridge Engineering Training Center of the Burma Construction Corporation. Through the actual construction of the Thuwana Bridge and the on-the-job training by the specialists, the country hopes to train and produce its own technicians.

The Thuwana Bridge will be built on the Ngamoyeik Canal which runs close to the Thuwana Bridge Engineering Training Center. It will connect the Thaketa District to the heart of Rangoon City upon its completion, and will help to relieve the increasing traffic between the two places. It is serves as the training site for the Bridge Engineering Training Center, Here, a few technical matters require special attention:

1. As an experimental practice for future bridge building on large rivers, it will be built as a long spanned, prestressed concrete bridge with a midspan over 100m.

2. The Cantilever Construction Method will be employed for the construction of the bridge because it can be used in any situation disregarding navigability or the currents of the river. The safe and accurate Dywidag Free Cantilever Method which is easy to operate will be used for the superstructure.

3. Three spannea continuous rigid frame with sliding hinges in central spann will be adopted because this structural type is most suitable for the Cantilever Construction Method.

4. For foundation of piers in the river, caisson foundations will be adopted in order to counteract the heavy piers.

5. The river, has a tidal range of 6m. and flows at a speed of 3-4 m/sec. To ensure the highest security and accuracy, the open caisson method using artificial island and temporary stage will be employed.

6. The jointed pre-cast reinforced concrets piles used presently in Burma will be abandoned since the are not competently earthquake-proofing. Instead, large diameter cast-in-place concrete piles will be employed. This technique can be adopted not only for this bridge but for any other kinds of construction as well.

7. Local resources and equipment should be utilized whenever possible. Concrete using local cement with a max. strength at $\sigma 28$ = 350 kg/cm^2 will be used for the superstructure applying σck = 350 kg/cm^2 in designing.

8. Simple and comprehensible calculation methods must be adopted in designing this bridge, since the method will be used in a text-book at the Bridge Engineering Training Center.

I DESIGN

CONTENTS

		Page
	Introduction	1-1
Chapter 1.	Basic Considerations	1-2
Chapter 2.	Spans of the Main Bridge and Dimension of Piers	1-8
Chapter 3.	Spans and Structures of the Simple Beam Bridges	1-13
Chapter 4.	Structural Type of Substructure	1-15
1.	General	1-15
2.	Type of Foundation Pile	1-15
3.	Determination of Type of Caisson Foundation	1-22
4.	Specific Values of Foundation Soil Properties for the Use in Caisson Design	1-24
Chapter 5.	Design of the 2-Span Continuous Box Girder P.C. Bridge	1-39
1.	General	1-39
2.	Design Condition	1-41
3.	Determination of the Sectional Form of the Main Girder	1-47
4.	Structural Design of Longitudinal Sections	1-54
5.	Structural Design of Cross Sections	1-84
6.	Analysis of Stress in the Structure Under Construction	1-94
Chapter 6.	Design of Simply Supported P.C. Composite Beam	1-96
1.	General	1-96
2.	Design Conditions	1-99
3.	Dimension	1-102
4.	Design Calculation	1-103
Chapter 7.	Design of Substructures	1-119
1.	Design Condition	1-119
2.	Design of Abutments	1-122
3.	Design of Piers	1-126

		Page
Chapter 8.	Design of Apputenances	1-135
1.	Bearing Devices	1-135
2.	Expansion Joints	1-139
Chapter 9.	Main Materials	1-143

Introduction

The engineering service for Thuwanna Bridge of Burma is undertaken as part of the oversea technical cooperation program of JICA, the Japan International Cooperation Agency, to provide detailed design for a prestressed concrete bridge spanning over Ngamoyeik Creek in Thuwanna Area of Rangoon City, Burma.

The construction of this bridge is intended also as an on-the-job training project for the Bridge Technical Training Center of Burma. For this reason, the central span of the bridge has been required to be not less than 100 m, to facilitate opportunities of training local technical personnels in design and construction of large span bridge.

Chapter 1. BASIC CONSIDERATIONS

1. Design Criteria

Prior to commencement of the design work, the following design criteria were established based on results of field reconnaissance and discussions with the Burmese authorities concerned.

(1) Type of Structure

The main bridge (superstructure) is to be a P.C. bridge constructed by cantilever erection method. The approach is to be made of a post tensioned T-beam bridge, Foundations are to be caisson and pile types.

(2) Width of Bridge Deck

The carriageway is to be 8.0m wide, and each of the sidewalks which are to be provided on both sides of the carrigeway is to be 1.5m wide.

(3) Longitudinal Gradient

The maximum slope, is not to be greater than 3%.

(4) Cross fall

The cross fall is to be 2% both for the carridgeway and for the sidewalk (Fig. 1.1).

Through inspections of locally existing bridges, it has been found that soil carried onto the bridge from the approach tends to plug the drain holes and cause the failure of the drainage system. In consideration of this, and also the fact that the longitudinal gradient of 3% is a rather flat one, a cross fall of 2% is proposed for this design.

(5) Alignment

The bridge spans are to be laid out in a straight line, whereas the alignment of the earthwork sections of the approaches are to be designed locally.

(6) Navigation Passage

Navigation passage is to be provided under the central span of the bridge with a width of 33.0m at minimum and a vertical clearance of 9.5m above highest high water level. (Fig. 1-2)

(7) Parvement

To be asphalt pavement of 5 cm thickness at minimum.

(8) Railing

The railing is to consist of a 80 cm high concrete slab parapet at the bottom and a fence on top. The fence is to be designed locally in Rangoon.

(9) Appurtenances

ϕ300 (1.0) 2 numbers for water piping

ϕ100 (1.0) 2 numbers for telephone and electricity wiring

(10) Specification applied

The Japanese Specification is applied.

(11) Line Load TL-20

(12) Seismic Coefficient

Basic seismic coefficient = 0.1, with k_h = 0.12.

(13) Temperature Change

Based on data provided by the Burmese Authority

$$T = \pm 15°C$$

(14) Wind Load

The 10 min. average wind velocity is to be taken as 40 m/sec.

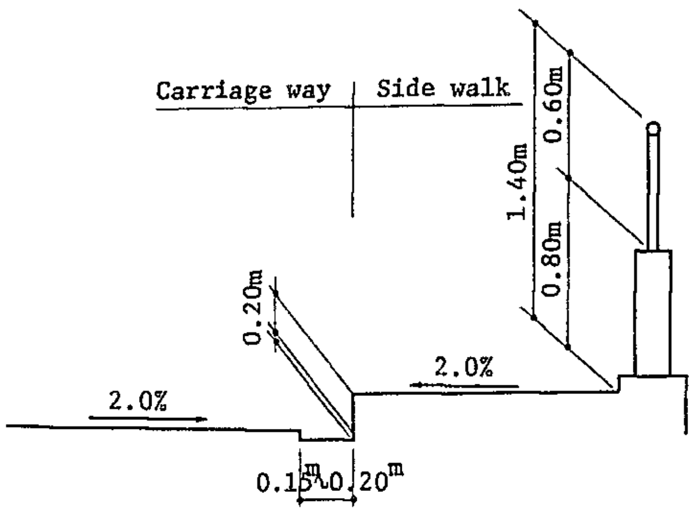

Carriage way
Side walk
0.60m
1.40m
0.80m
0.20m
2.0%
2.0%
0.15m~0.20m

Fig. 1-1

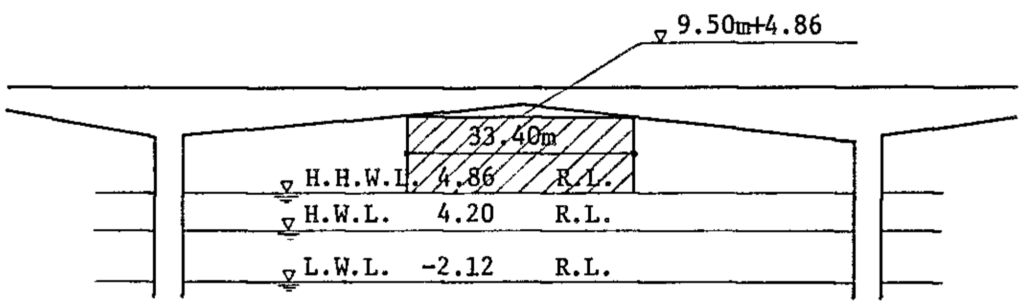

9.50m+4.86
33.40m
H.H.W.L. 4.86 R.L.
H.W.L. 4.20 R.L.
L.W.L. -2.12 R.L.

Fig. 1-2

2. Site Conditions

(1) River Crosssection and Bridged Length

River crosssection at the bridge site is as shown in Fig. 1-6. The overall length of the bridgework is to be 300m, approximately equal to the length of the existing bridge.

(2) Water Levels

Data for water levels, as illustrated in Fig. 1-2, are as follows:

H.H.W.L. = 4.86 RL
H.W.L. = 4.26 RL
L.W.L. = -2.12 RL

(3) Soil Condition

As soil data for the project site is not yet available, considerations are made based on data for the site of the existing bridge locating in the vicinity, given in "Report on Subsurface Investigation of Nga-Moe-Yeik Change Bridge Site, Tsuwanna, Rangoon, Burma".

3. Structural System

(1) Structural Type of the Main Bridge

The rational type of structure for a P.C. girder bridge to be constructed by cantilever erection method is one with the horizontal member built integrally from the supporting pier in rigid frame. To relieve the structure from the influences of temperature change, drying shrinkage, and contraction due to prestressing, it is desirable that the central span be built with a horizontally movable hinge.

Based on the above points of consideration, and also the fact that the central span is required to be not shorter than 100m as previously stated, a 3-span rigid framed continuous P.C.

girder with horizontally movable central hinge is adopted as the basic form of the main bridge structure. (Fig. 1-3)

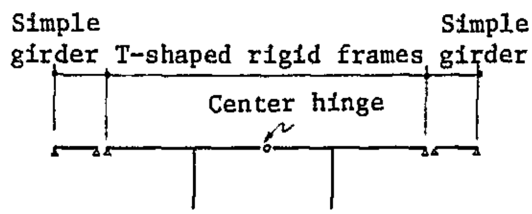


Fig. 1-3

(2) Road Section

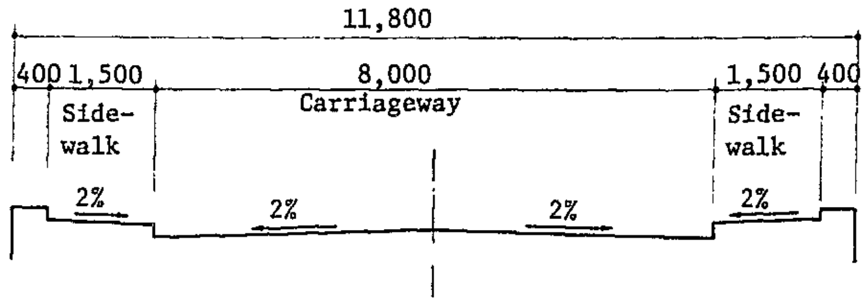


Fig. 1-4

(3) Profile of Bridge Pavement

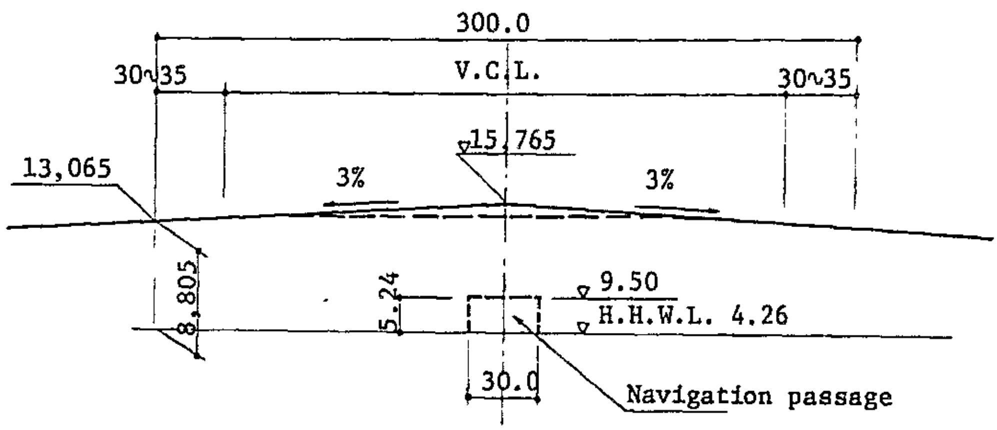
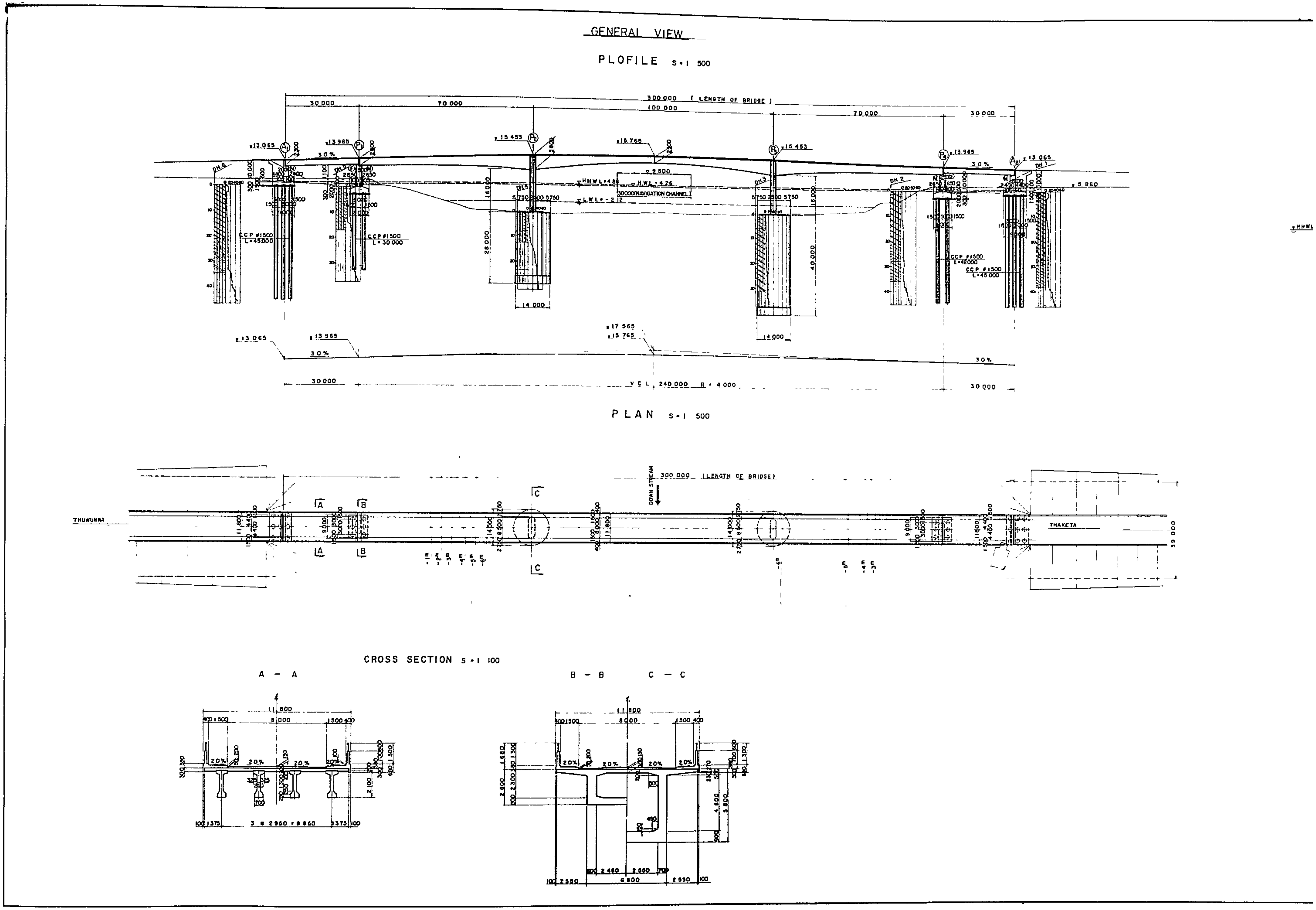
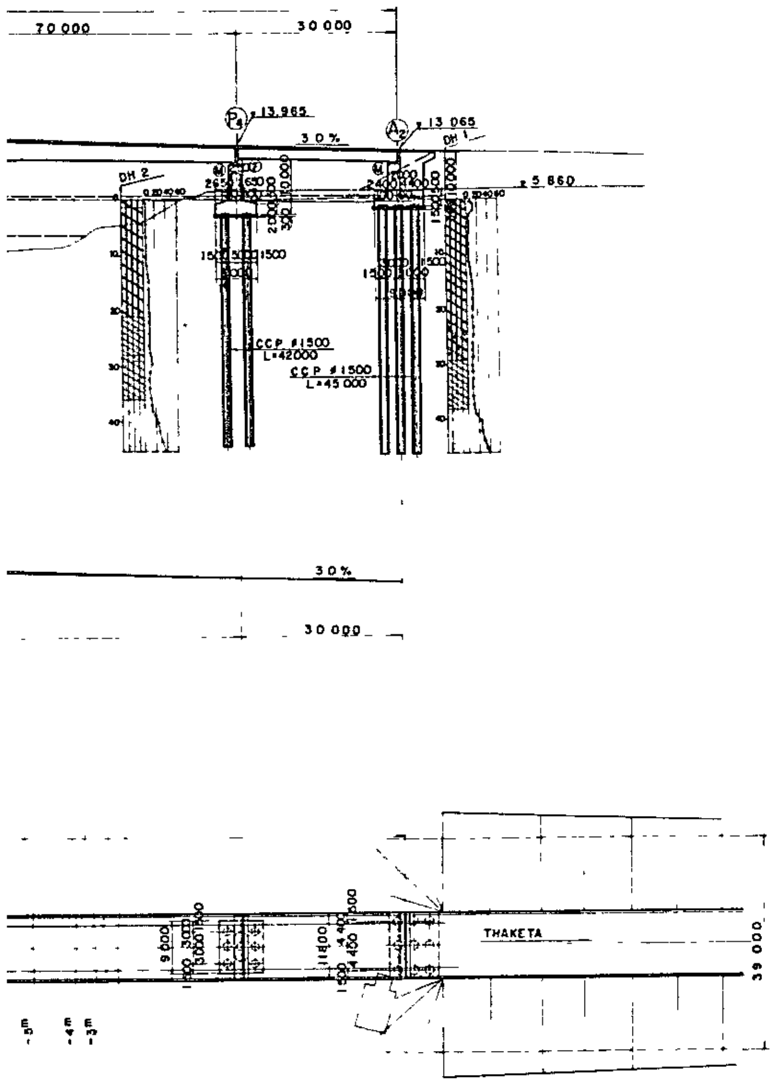
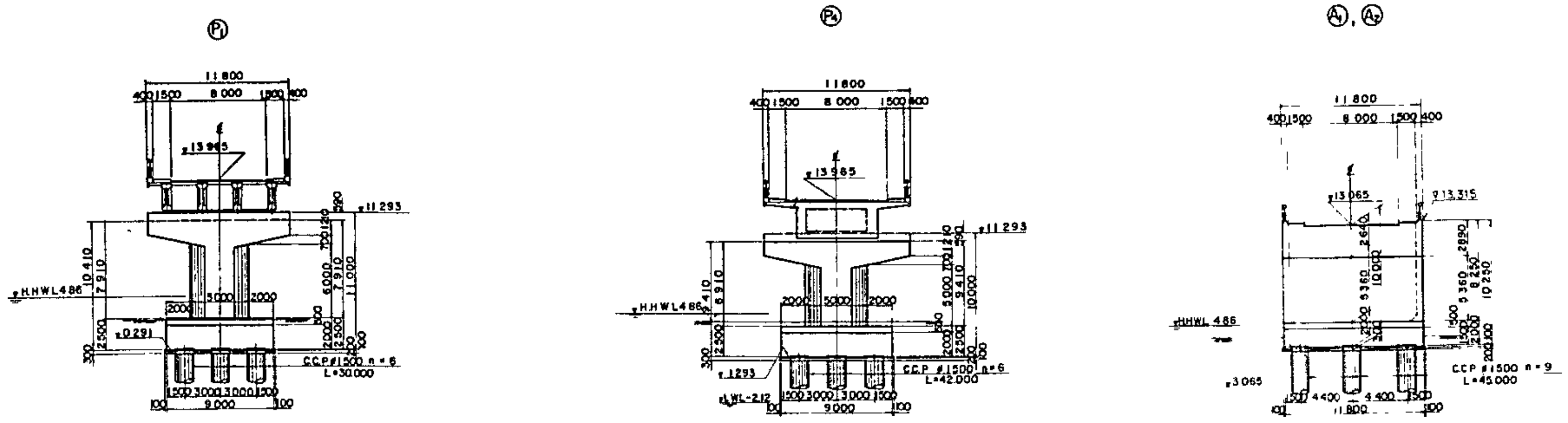


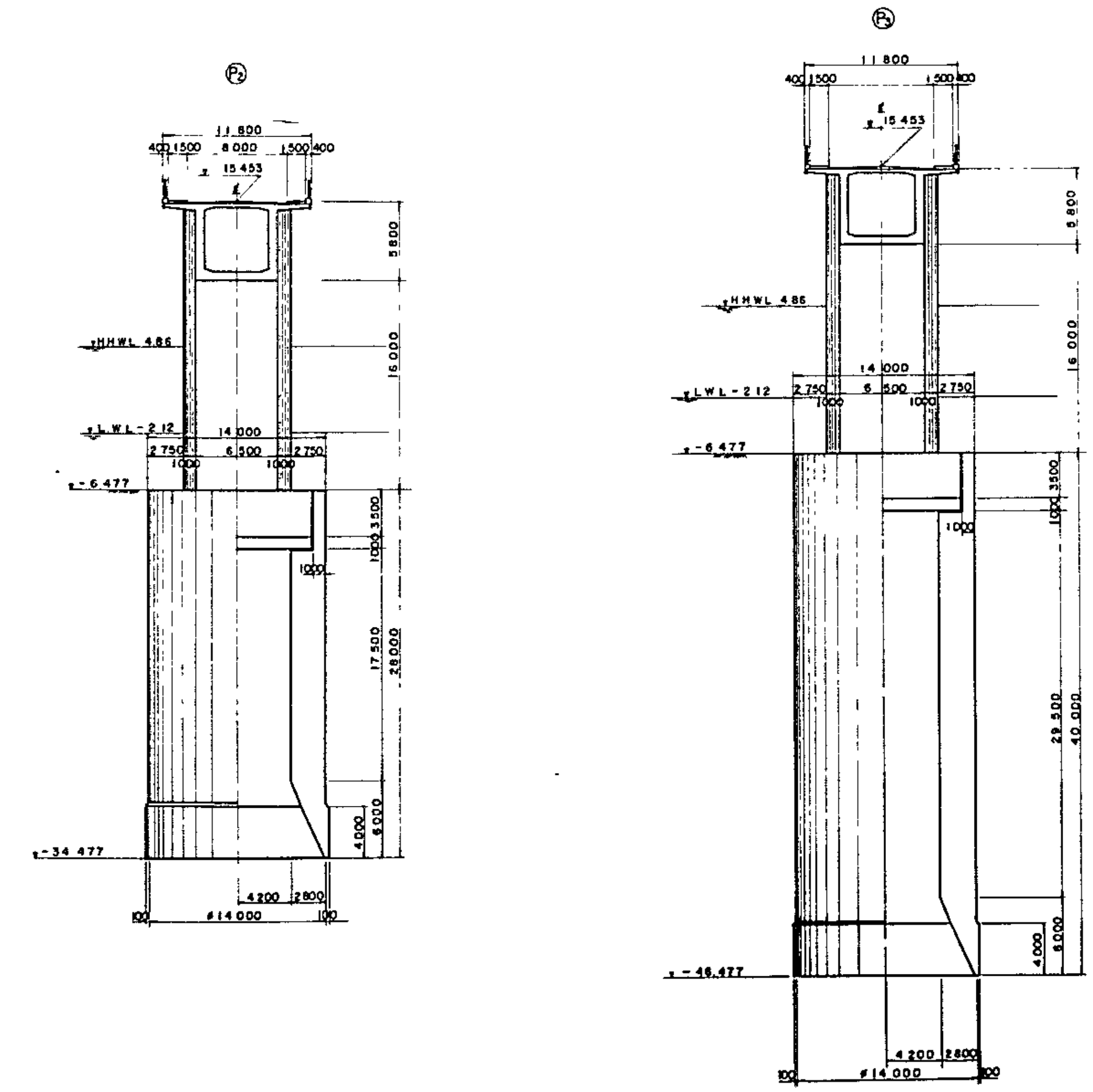
Fig. 1-5


GENERAL VIEW
PLOFILE S=1 500
300 000 (LENGTH OF BRIDGE)
100 000
70 000
30 000
30000 (NAVIGATION CHANNEL)
CCP φ1500 L=45000
CCP φ1500 L=30 000
CCP φ1500 L=42000
CCP φ1500 L=45000
14 000
VCL 240 000 R=4 000
PLAN S=1 500
300 000 (LENGTH OF BRIDGE)
DOWN STREAM
THUWUNNA
THAKETA
CROSS SECTION S=1 100
A — A
B — B
C — C
11 800
8 000
3 @ 2950 = 8 850



CROSS SECTION s = 1:200





DESIGN DATA

LENGTH OF BRIGE	TOTAL LENGTH 300m
GIRDER LENGTH	29.85m+69.95m+100m+69.95m+29.85m
LIVE LOAD	TL-20
TYPE OF STRUCTURE	PRESTRESSING CONCRETE BRIDGE BY CANTILEVER CONSTRACTION METOD PRESTRESSED CONCRETE GIRDER
SPAN	29.15m+69.5m+100m+69.5m+29.15m
WIDTH OF CARRIAGEWAY	8m CARRIAGEWAY WITH 1.5m SIDEWALK ON BOTH SIDES
TRANSVERSE GRADIENT	FOR CARRIAGEWAY 2.0% FOR SIDEWALK 2.0%
LONGITUDINAL GRADIENT	MAXIME 3.0 %
DESIGN SEISMIK COEFFICIENT	Kh = 0.12
STRENGTH OF CONCRETE MAIN GIRDER	350kg/cm

THUWUNNA BRIDGE
DETAIL ENGINEERING SERVICE

GENERAL VIEW

Scale 1:500, 1:200 | Sheet No in | Date Mar. 1980

Designed by CHIYODA CONSULTANT CO., LTD

JAPAN INTERNATIONAL COOPERATION AGENCY

Chapter 2. SPANS OF THE MAIN BRIDGE AND DIMENSION OF PIERS

Since the total length of the bridgework, as previously stated, is to be approximately 300m, and the main bridge is to have a central span of 100m length or more, and to have a simply supported beam of about 30m length extended from each end of it, the range of possible varieties of alternative design for the bridge spans proportioning is limited.

Major aspects of consideration in determining the span proportioning are as follows.

a) Economical aspect : The shorter the length of the large span structure, the more economical it will be.

b) Ease of execution : Construction of a side span longer than the cantilever of central span will require the use of staging at its outer end, which should desirably be as short as possible.

Piers on both ends of the main bridge, on the other hand, should desirable be so designed that construction works can be done on the river bank.

c) Structural requirement: Piers should not be always subject to the stress of bending moment.

Based on the above points of consideration, the following alternative span designs are compared.

Alternative 1. 67 + 100 + 67 = 234 m
2. 75 + 100 + 75 = 250 m
3. 70 + 100 + 70 = 240 m

Each of these have the following feature respectively.

1. Alternative 1: Bridge piers at both ends of the main bridge can be constructed on the river bank. The total bridge length is short, and no staging is required for the outer end portion the side spans.

The bridge piers however are constantly subject to considerably large bending moment from the dead load.

2. Alternative 2: No bending moment is received by the bridge piers from dead load.

   However, long outer end portions of side spans require staging during construction works, and the total length of the structure is also much larger.

3. Alternative 3: Has an intermediate nature between the other two alternatives in all aspects stated above, Alternative 3 is chosen for the bridge design, as a result of the above evaluation (Fig. 2-1).

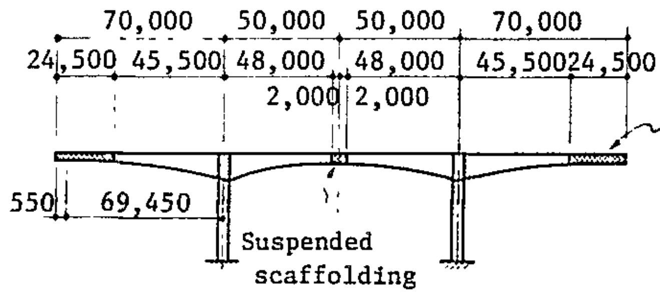


Fig. 2-1

2. Length and Section of Piers

A caisson designed with crown level at the H.W.L. would be easy to construct, but on the other hand, would expose itself during the time of L.W.L. and spoil the view. Also, where the foundation involved is massive, being so designed would be an unfavorable factor against seismic load. In view of these, crown levels of the caissons will designed approximately at the river bed level.

As previously stated, alternative 3 of span proportioning designs for the main bridge, of which piers will constantly receive bending moment from the dead load, is adopted for the design. This bending moment can be decreased by lowering the rigidity ratio (= moment of inertia of section/length). For this purpose, 4 varieties of bridge pier dimensions are taken for comparison of the bending moment, as in the following tables.

Variety	Height of pier	Thick-ness of pier	Moment of inertia of section	Rigidity
A	15,690	2.5m	9,277	0.591
B	15,690	3.0m	15,134	0.965
C	15,690	4.0m	34,308	2.187
D	4,200	4.0m	33,070	7.874

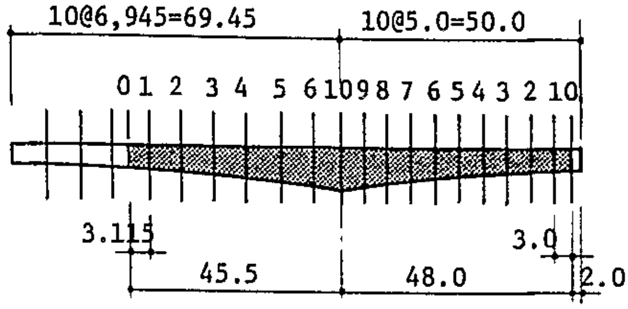
[t.m.]

Alternative	At completion				During construction
	Dead load	Live load	Total	Under seismic force	
A	1,446.7	1,667.7	3,114.4	5,267.3 -3,904.9	2,872.0
B	2,047.1	2,360.3	4,407.7	5,478.0 -3,624.3	2,872.0
C	3,160.9	3,643.9	6,804.8	5,878.0 -3,502.5	2,872.0
D	2,960.9	5,357.4	8,318.3	3,232.0 1,365.9	3,286.0

Conclusions:

Of the above 4 varieties, A sustains the least bending moment in the piers and is aesthetically the best in proportion. It is therefore adopted for the bridge design.

3. Imbalance moment during Construction



	A	W	W	S	S· X	Mg	Mw	M
0	8,444	21.11	65.8	65.8	205.0	0		
1	8,697	21.74	151.1	216.9	1,506.4	205		
2	9,030	22.58	156.9	373.8	2,596.0	1,711		
3	9,434	23.59	164.8	538.6	3,740.6	4,307	41.5x50	
4	10,488	26.22	183.3	721.9	5,013.6	8,048	= 2,075	
5	12,360	30.90	214.4	936.3	6,502.6	13,062		
6	14,073	35.18	243.7	1,180.1	8,195.8	19,564		
7	15,371	38.43	129.9	1,309.9	9,097.3	27,760	2,075	29,835
10	15,371	38.43	93.9	1,195.2	5,976.0	24,788	2,175	26,963
9	14,319	35.80	178.9	1,101.3	5,506.5	19,281		
8	13,205	33.01	164.4	922.4	4,612.0	14,669		
7	11,498	28.75	143.7	758.0	3,790.0	10,879	435x50	
6	9,764	24.41	122.2	614.3	3,071.5	7,808	= 2,175	
5	8,178	20.45	102.4	492.1	2,460.5	5,347		
4	6.979	17.45	88.3	389.7	1,948.5	3,399		
3	6,817	17.04	85.2	301.4	1,507.0	1,892		
2	6,698	16.75	83.8	216.2	1,081.0	811		
1	6,625	16.56	82.9	132.4	662.0	149		
0	6,601	16.50	49.5	49.5	148.5	0		

Imbalance moment M = 2,872.0 t.m

Chapter 3. SPANS AND STRUCTURES OF THE SIMPLE BEAM BRIDGES

Since the total length of the bridgework is to be 300 m, and that of the main bridge, 240m, the spans of simple beam bridges to be added to both ends of the latter will naturally be 30m each (Fig. 3-1).

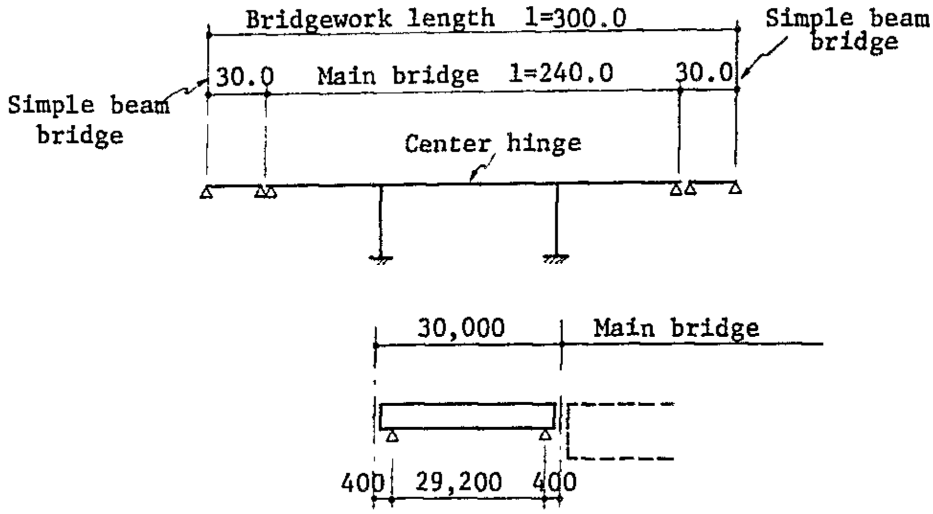


Fig. 3-1

Two commom types of structure exist for the construction of a prestress concrete simple beam bridge of 30m span.

These are:

1. Simply supported P.C. T-beam
2. Simply supported P.C. composite beam

In the recent trend, P.C. composite beam is the more often constructed type, being easier to execute compared to P.C. T-beam. The height of a main beam section is normally designed roughly as in the following, taking not only strength into consideration but also the aesthetical effect of its proportion against the span length.
(in this case = 29.2m)

For P.C. T-beam: $h = L \times 1/18 = 1{,}620m$

For P.C. composite beam: $h = L \times 1/16 - 15 = 1{,}825 - 1{,}950$

From economical point of view, however, a relatively larger height of the beam section would be better. In view of this, the height of the P.C. composite beam which is adopted in this design, is taken at h = 2,100m. Its proportion against the span length is therefore: = 2.1/29.1 = 1/13.9.

Crosssections of the simple beam bridge is as illustrated in Fig. 3-2, 3-3 below.

(1) Simply Supported P.C. Composite Beam Bridge

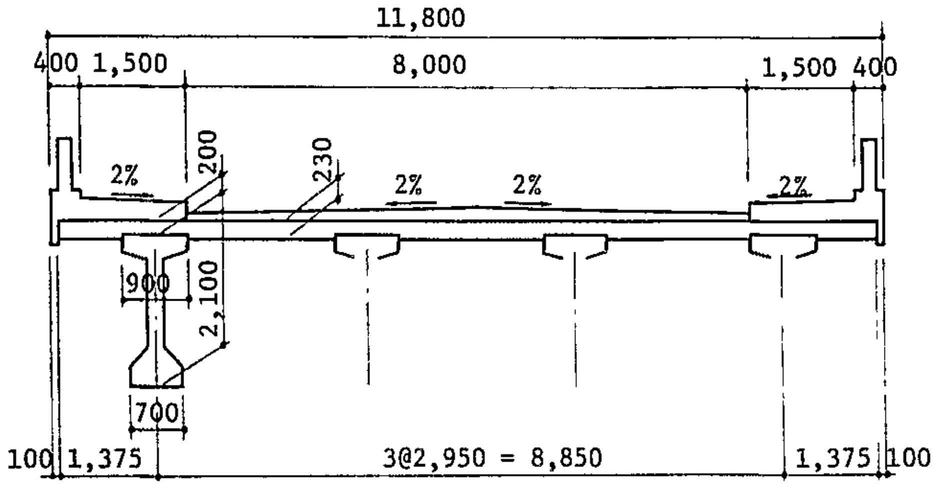


Fig. 3-2

(2) Simply Supported P.C. T-beam Bridge

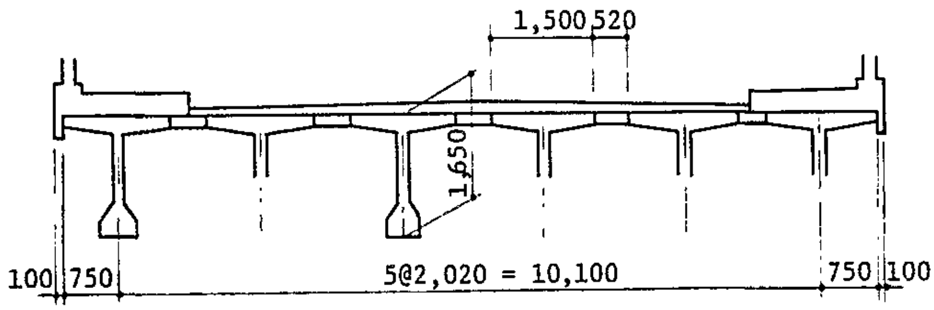


Fig. 3-3

Chapter 4. STRUCTURAL TYPE OF SUBSTRUCTURE

1. General

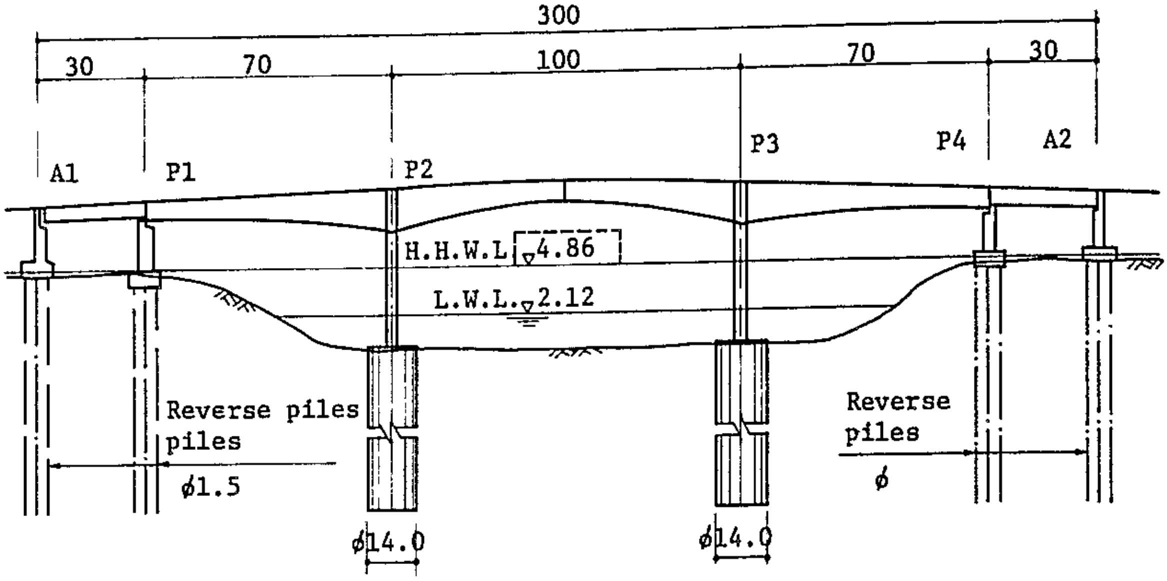


Fig. 4-1

It is a provided condition that A1, A2, P1 and P2 be constructed with pile foundations, while piers P2, P3, with caisson foundations. This chapter deals with the determination of type of pile, depth of foundation embedment, and method of determining the specific values of foundation soil properties to be used in caisson design.

2. Type of Foundation Pile

(1) Alternative types of piles and design criteria

Study design of foundations using the following alternative types of pile are made for comparasion to determine the type of pile to be adopted in the design.

Alternative 1. Precast R.C. pile of 400 x 400 square section
2. Cast-in-situ R.C. pile, ϕ1,500
3. Cast-in-situ R.C. pile, ϕ1,200

Design criteria for the study design are as follows:

a) Concrete

Item		6ck(kg/cm^2)	6cal(kg/cm^2)
Abutment A1, A2	The pier	210	70
	Footing	210	70
Piers P1, P4	The pier	210	70
	Footing	210	70
Piers P2, P3	Wall/post	300	100
	Top slab of caisson	240	80
	The caisson proper	180	60

b) Reinforcement SD30

Allowable Stress: Undernormal conditon σsa = 1800 kg/cm^2

Under seismic load σsa = 2700 kg/cm^2

c) Foundation pile

Strength of concrete:

For R.C. pile: 6ck = 210 kg/cm^2

For cast-in-situ pile: 6ck = 300 kg/cm^2

d) Allowable amount of displacement

	Under normal conditon	Under seismic load
Displacement method:	15	25 mm
Conventional method:	10	15 mm

The following methods exist for the calculation of the bearing strength of foundation piles.

1 "Specification for Design of Road Bridge Substructures - Division of Pile Foundation"

2 Terzaghi Formula

3 Meyerhof Formula

In this design method 1, which is commonly used in Japan is applied.

(2) R.C. Pile

a) Cross section: 400 x 400

b) Strength of concrete: σ_{ck} = 210 kg/cm^2

σ_{ca} = 70 kg/cm^2

c) Reinforcement layout

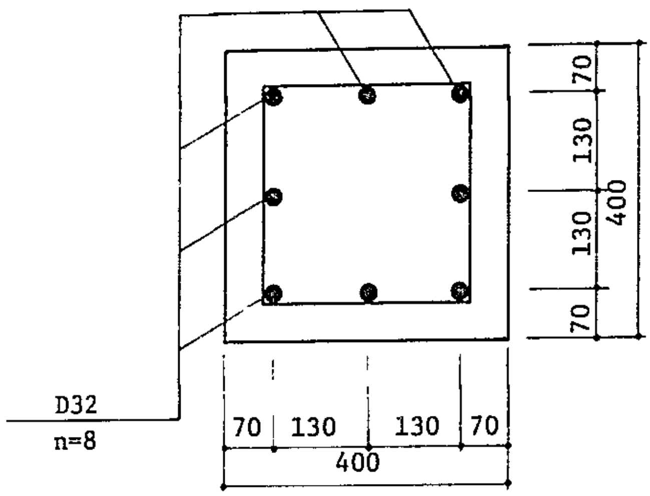


Fig. 4-2

d) Allowable Bearing strength

Length of pile: ℓ=42,000^m

Under normal condition: q_a = 89.8 ≒ 80.0 t/pile

Under seismic load: q_e = 119.7 ≒ 110.0 t/pile

e) Determination of the number of piles

Taking the pier P4 as an example, the layout of piles is as snown in the following figure.

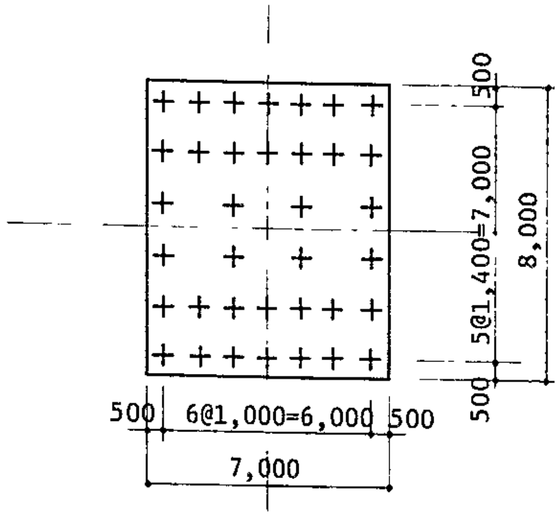


Fig. 4-3

Item		Unit	Design value	Allowable value	Remarks
Verticle reaction	R max	t/pile	62.9	80.0	Determined based on normal condition
	R min		25.8		"
Horizon-tal force	Rh	t	4.1		
Displace-ment	δ	mm	4.0	10	
Pile head moment	Mt	tm	5.27		
Stresses in the pile	M	"	5.27		
	N	t	62.9		
	H	"	4.1		
	As	cm^2	63,536		
	δc	kg/cm^2	67.41	70	
	σs	"	8.73	1600	
	τ	"	23	6.5	

(3) The ϕ1200 cast-in-situ R.C. pile

a) Cross section: 1200ϕ

b) Strength of concrete: $\sigma ck = 300\ kg/cm^2$

$\sigma ca = 80\ kg/cm^2$

c) Allowable bearing strength

Length of pile: ℓ = 42,000 m

Under normal condition: qa = 287.9 ≒ 280 ton per pile

Under seismic load: qe = 431.9 ≒ 430 ton per pile

The bearing strength is calculated based on the Japan Road Association Specification.

Pile bottom N value: N = 30 or more

Skin friction

Sandy soil: $\frac{N}{2}$ ($\leqq$12)

Cohesive soil: $\frac{C}{2}$ or $\frac{N}{2}$ ($\leqq$12)

d) Determination of the number of piles

As the required number of piles is to be determined on the basis of the normal range of temperature change, studies in this aspect will be made bearing the normal range of temperatue change in mind.

The minimum interval of pile spacing is taken at 2.0 m.

Taking pier P4 as an example, the layout of piles will be as in the following figure.

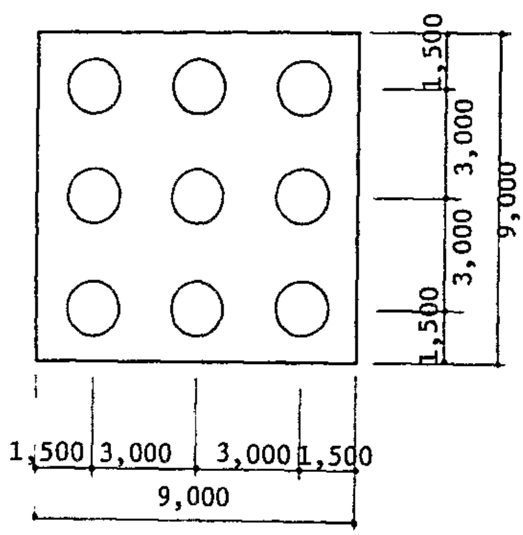


Fig. 4-4

Item		Unit	Design value	Tolerable value	Remarks
Verticle reaction	R max	t/pile	286.4	288.0	Determined based on normal condition
	R min		91.3		
Horizontal force	Rh	t	16.4		
Displacement	δ	mm	6.2	10	
Pile head moment	Mt	tm	27.6		
Stress in the pile	M	"	27.6		
	N	t	91.3		
	H	"	16.4		
	As	cm^2	0.79 n=12 46.45		Determined based on the min. reinforcement
	σc	kg/cm^2	26.1	80	
	σs	"	142	1600	
	τ	"	0.5	8	

(4) The 1,500 cast-in-situ R.C. pile

a) Cross section: 1,500ϕ

b) Strength of concrete:

$\sigma ck = 300 \ kg/cm^2$

$\sigma ca = 80 \ kg/cm^2$

c) Allowable bearing strength:

Length of pile: ℓ = 42,000 m

Under normal condition: qa = 395.6 ≒ 395 ton per pile

Under earthquake: qc = 593.4 ≒ 590 ton per pile

Bearing strength is calculated based on the Japan Road Association Specification.

Pile bottom N value: N = 30 or more

Skin friction:

Sandy soil: $\frac{N}{2}$ ($\leqq 12$)

Cohesive soil: $\frac{e}{2}$ or $\frac{N}{2}$ ($\leqq 12$)

d) Determination of the number of piles

As the number of piles is to be determined in consideration of the normal range of temperature change, all studies in this regard will be done bearing the normal range of temperature change in mind.

Taking Pier P4 as an example, the layout of piers will be as in the following figure.

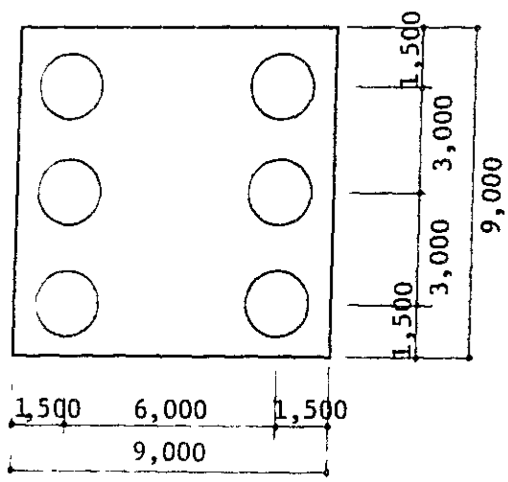


Fig. 4-5

Item		Unit	Design value	Tolerable value	Remarks
Verticle reaction	R max	t/pile	383.68	395.0	Determined based on normal condition
	R min		182.85		
Horizontal force	Rh	t	24.62		
Displacement		mm	6.3	10	
Pile head moment	Mt	tm	50.07		
Stress in the pile	M	"	50.07		
	N	t	182.85		
	H	"	24.62		
	As	cm^2	D22 n=20 77.42		
	σc	kg/cm^2	24.2	80	
	σs	"	35.2	1600	
	τ	"	0.2	8	

(5) Conclusion

Based on the above results of study design, the characteristics of the alternative types of piles are respectively as follows.

a) R.C. pile

The length of pile will be ℓ = 42,000 mm and driving of such a pile will be a difficult job.

Reliability of the joint (3 - 4 numbers for a pile) is doubtable.

Strength of each individual pile is small.

b) 1200ϕ Cast-in-situ Pile

An increase of the number of pile by 50% will be required compared to the case 1500ϕ ones.

Having less margin of bearing strength compared to the case of 1500ϕ ones.

c) 1500ϕ cast-in-situ piles

Cost of casing is higher than the case of 1200ϕ ones.

Greater amount of concrete will be required.

All the three alternatives types of pile have their negative features respectively. Among these, R.C. pile involves uneasy factors in engineering aspect and the use of it is therefore not desirable.

As for the cast-in-situ piles, possibility of reemployment of the piling machine for ϕ1,500 ones at other project sites of similar soil conditions in future is considered higher than that of the other case. The use of cast-in-situ pile of ϕ1,500 size is therefore selected.

3. Determination of Type of Caisson Foundation

(1) Assumption of Execution Method

The following two common methods exist for construction of caissons, both having certain benefits and disadvantages respectively.

a) Island method (employing landing stages)

b) Floating method (using barges)

For this project, however, as the disadvantages of the floating method, which include the following are likely to become very unfavorable factors causing.cast and time schedule problems, adaptation of the island method is assumed here.

a) Setting of the execution facilities is likely to be difficult due to the high velocity of the river flow.

b) Numerous specialized skilled workers are required.

c) Very large cutting edge is required.

(2) Shape of the Caisson

The sectional area and shape of a caisson vary depending on the position of its crown level. (Fig. 4-6)

a) Where the crown level is above the HWL:

- Pedestal moment (inbalance moment at the lower end in the column)
  Mo = 8318.33 t.m
- Required area of section (assumed to be rectangular)
  18.00 x 22.00 m

b) Where the crown level is at the level of the river bed:

- Pedestal moment:

  Mo = 5935.22 t.m
- Required area of section (rectangular)
  14.00 x 15.00 m

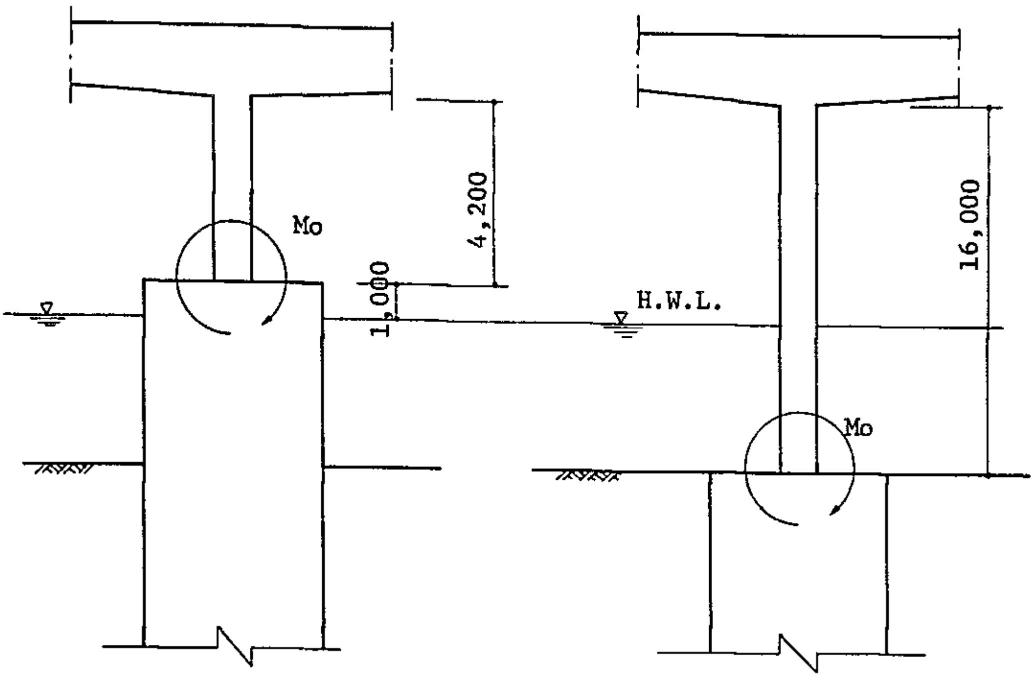


Fig. 4-6

(a)		(b)	
Section:	18.00 x 22.00	Section	14.00 x 15.00
Ground reaction	60.7 t/m^2 <70 t/m^2	Ground reaction	48.3 t/m^2 <70 t/m^2
Lateral reaction	5.6 t/m^2 <15 t/m^2	Lateral reaction	13.4 t/m^2 <15 t/m^2

As seen in the above table, case (a) is very uneconomical compared to case (b). Crown level of the caisson will therefore be designed at the level of the river bed. Since the work will be executed in the island method, the caisson will be designed with a circular section for the comparative case in sinking operation.

4. Specific Values of Foundation Soil Properties for the Use in Caisson Design

(1) Design Criteria

a) Soil mechanical values

The following values for the mechanical characteristics of the soil are estimated based on results of standard penetration tests, due to the lack of existing soil test data.

Method of deformation: $Eo = 28.N$
Angle of internal friction: $\phi = \sqrt{15N} + 15$

b) Allowable displacement at the caisson crown level.
Under normal condition: $\sigma a = 10$ mm
Under seismic load: $\sigma a = 20$ mm

A larger amount of displacement is allowed taking into consideration the margin of safety already provided by disaccounting the effect of soil cohesion, etc. in the analysis of the lateral bearing strength.

c) The anti-subsidance force of frictional resistance

Determination of the caisson section is made based on an anti-subsidence force of frictional resistance which is taken 2 - 2.5 times as large as the value of skin friction normally given in design specifications.

Such safety margin is made in view of the fact that the depth of sinking operation is great, and that in the event of difficulty in sinking of the caisson due to any unforeseeable cause of resistance, the acquisition of additional load and the mobilization of equipment and skilled workers for the acceleration of the operation will likely be difficult.

d) A depth of 5.00 m of the river bed soil will not be accounted considering the possible effect of scouring (Fig. 4-7).

e) For the bearing soil, fine sand layer with N = 40 will be selected.

f) In calculating the allowable stress of horizontal ground reaction, effect of the soil cohesion will be ignored.

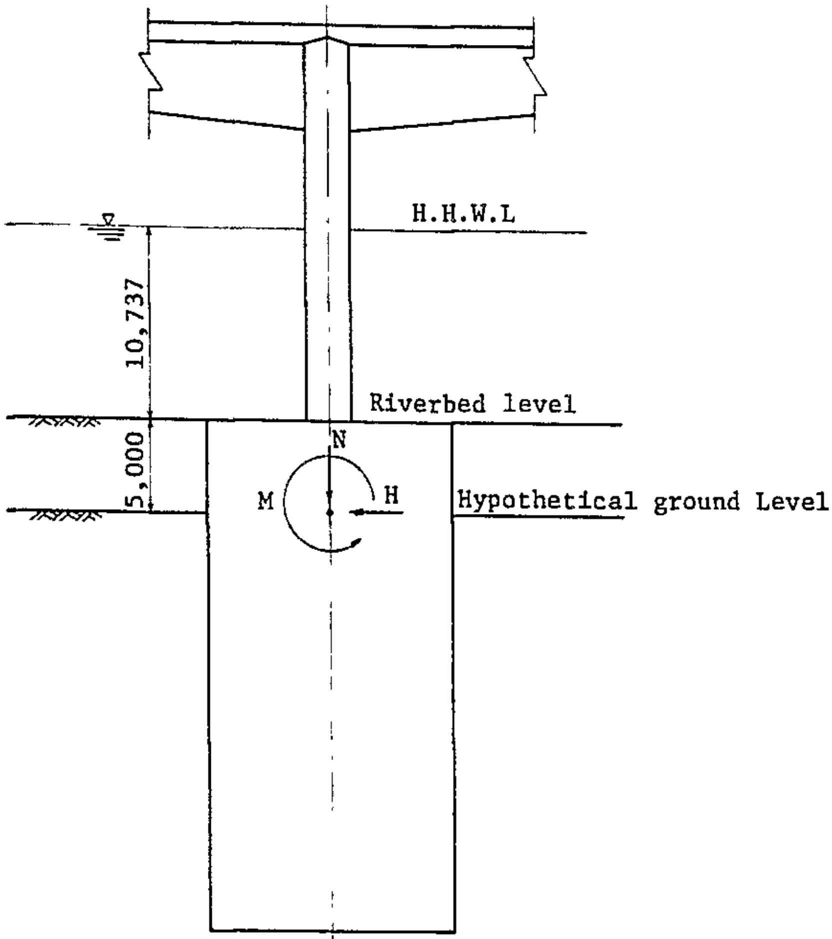


Fig. 4-7

(2) Ground Reaction Modulus

Due to the deficiency of existing soil test data, this calculation will be made on the cases of standard penetration test result (N value).

a) Horizontal ground reaction modulus

$$Kh = Kho \left(\frac{Bh}{30}\right)^{-\frac{3}{4}} = 12.8 Kho Bh^{-\frac{3}{4}}$$

where,

Kh: Horizontal ground reaction modulus (kg/cm^3)

Kho: Horizontal ground reaction coefficient (kg/cm^3) equivalent to the result of plate bearing test using a rigid disc of 30 cm diameter, formulated as follows inclusive of the 20% side surface component.

$$Kho = \frac{1.2}{30} \alpha Eo = 0.040 \alpha Eo$$

Bh: Equivalent loaded width of the foundation (in cm), obtained from the following formula:

$$Bh = \sqrt{Ah}$$

Eo: Modulus of deformation of the ground (kg/cm^2) at the project location, determined or estimated based on the methods given in table 4.

α: Coefficient which varies between cases under normal conditions and cases under seismic load, as in following table.

Ah: Frontage area of the caisson (cm^2)

Method of which the modulus of deformation Eo is obtained Eo (kg/cm^2)	α	
	Under normal condition	Under seismic load
1/2 of the modulus of deformation obtained by using formula (1) based on the curve of results of repeated plate bearing tests using a rigid disc of 30 cm diameter	1	2
Obtained from bore holes	4	8
Obtained through unconfined compression test or triaxial compression test of test sample	4	8
Estimated by substituting the N value of standard penetration test result into the formula Eo = 28N	1	2

b) Verticle ground reaction modulus

$$Kv = Kvo \left(\frac{Bv}{30} \right)^{-\frac{3}{4}} = 12.8\ Kvo\ Bv^{-\frac{3}{4}}$$

Where,

Kv: Verticle ground reaction modulus (kg/cm^3)

Kvo: Verticle ground reaction coefficient (kg/cm^2) equivalent to the value of plate bearing test using a rigid disc of 30 cm diameter, obtain by applying the following formula:

$$Kvo = \frac{1}{30}\alpha Eo - 0.033\,\alpha Eo$$

Bv: Equivalent loaded width (cm) of the foundation obtained from the following formula or the diameter of the disc in the case where it is used.

c) Horizontal shear modulus

$Ks = \lambda . Kv$

Ks: Horizontal shear modulus (kg/cm^3)

λ: Ratio of the horizontal shear modulus against the verticle ground reaction modulus. Its value ranges between 1/2 and 1/5 according to test results. However, in the calculation system for the computation of the deformation, greater values are taken, around λ= 1/3 - 1/4 so as the value of Ks falls on the safe side, yielding larger computed amount of displacement.

(3) Allowable vertical bearing stress of the caisson bearing soil

o The allowable vertical bearing stress of the subsoil of the caisson shall be the value resulting from dividing the ultimate bearing stress of the subsoil obtained by applying statical formula, by the following safety factors.

Under normal condiotn:	3
During earthquake:	2

δ The allowable bearing stress of the subsoil based on statical formula is formulated as follows:

$$qa = \frac{1}{n}(qd - \gamma_2 Df) + \gamma_2 Df$$

$$qd = \alpha cNc + \frac{1}{2}\beta\gamma_1 BN\gamma + \gamma 2DfNq$$

Where;

qa: allowable bearing stress of the caisson subsoil (t/m^2)

qd: Ultimate bearing stress of the caisson subsoil (t/m^2)

c: Cohesive force of the soil below the bottom level of the caisson (t/m^2)

$\gamma 1$: Unit weight of soil below the bottom level of the caisson or underwater unit weight in the case of soil below ground water table (t/m^3)

$\gamma 2$: Unit weight of the circumferential soil above the bottom level of the caisson or underwater unit weight where it is below the ground water table. (t/m^3)

α, β: Shape factor of the caisson bottom surface as given in the table below.

Shape factor of the caisson bottom	Narrow strip	Square	Oblong or oval	Circle
α	1.0	1.3	$1 + 0.3\frac{B}{L}$	1.3
β	1.0	0.6	$1 - 0.4\frac{B}{L}$	0.6

Here, L is the frontage width (m) of the caisson and B/L >1, B/L is taken as 1 instead.

B: Side area of caisson (m), or diameter of caisson where it is a circular cylinder.

D_f: Effective depth of embedment (m) between the hypothetical ground level and the bottom level of the caisson, representing the depth of embedment below the future ground level, where changing of ground surface due to scouring etc. is anticipated.

N_c, N_q, N_γ: The value of the bearing power modulus is as illustrated in the following figure.

n: Safety factor

ϕ: Angle of internal friction

(4) Horizontal bearing stress

The value resulted from dividing the stress of passive earth pressure in the soil by the following safety factors wii be adopted as the horizontal bearing power at that location.

o Safety factor:

Under normal condition: 1.5
Under seismic load: 1.1

o Earth pressure formulas:

i) For sandy soil

$$Pp = \gamma KpX + Kp \cdot q$$

ii) For cohesive soil

$$Pp = \gamma KpX + 2c\sqrt{Kp} + Kp \cdot q$$

iii) For cases under seismic load

$$Pp = (1 + Kv)\,\gamma x K_{EP} + 2c\sqrt{K_{EP}}$$

where,

Pp: Stress of passive earth pressure at point with a depth x. (t/m^2)

q: Ground surface load under normal condition (t/m^2)

c: Cohesive force of soil (t/m^2)

γ: Unit weight of soil, or underwater unit weight of soil where it is below ground-water table. (t/m^3)

x: Depth below ground level (m)

o Coefficient of earth pressure

Kp: Coefficient of passive earth pressure based on Coulomb earth pressure, calculated by using the following formula.

$$K_P = \frac{\cos^2\phi}{\cos\delta\left[1 - \sqrt{\frac{\sin(\phi - \delta)\sin\phi}{\cos\delta}}\right]^2}$$

K_{EP}: Coefficient of passive earth pressure under seimic load calculated by using the following formula.

$$K_{EP} = \frac{\cos^2(\phi - \theta_0)}{\cos\theta_0\cos(\theta_0 - \delta)\left[1 - \sqrt{\frac{\sin(\phi - \theta_0)\sin(\phi - \delta)}{\cos(\theta_0 - \delta)}}\right]^2}$$

kv: Vertical seismic coefficient, with vertically downward reactions denoted in positive values.

Kh: Horizontal seismic coefficient

ϕ : Angle of internal friction of the soil

δ : Angle of friction between the caisson wall surface and the soil.

θ_0: $\tan\theta_o = \frac{Kh}{1 + Kv}$

(5) The allowable vertical bearing power

(t/m^2)

	Design value	Maximum value given in reference literature	Adopted value	Remarks
Under normal condition	301.9	100	80	
During earthquake	442.6	150	120	

As considerable dispersion is observed among the data of bore holes No. 1 to No. 6 for the depth of bearing soil and the N value of the fine sand layer, small values at 80% of those given in relevant literature will be adopted for the allowable bearing power to allow for margin of it.

* For abstracts of reference literature regarding allowable bearing power, see (3).

(6) Subsoil bearing stress for design of caisson

This paper presents a proposed method of determining the soil bearing stress (under normal condition) for the use in the design of caisson foundation on sandy soils.

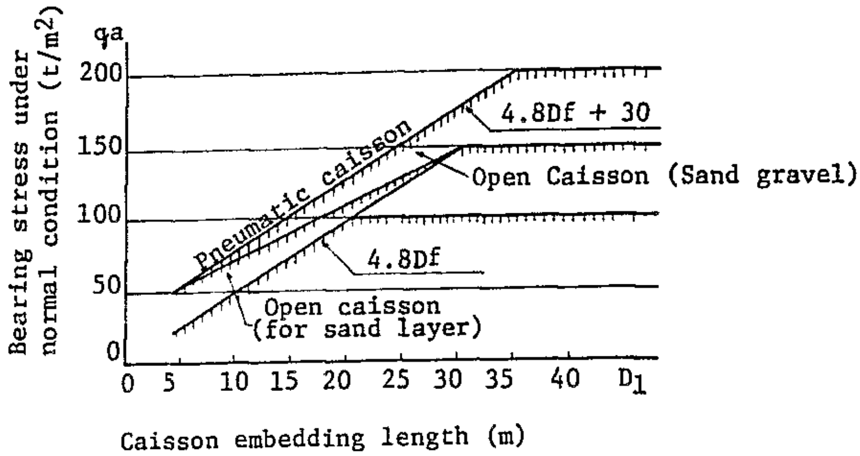
Basically:

Bearing Stress for Design Use (under normal condition)

No.	Normal bearing stress	0 100 200 300(t/m^2)
1	Type of caisson	Open caisson Pneumatic caisson
2.	Plate bearing test in pneumatic caisson	qd'(Effect of embedment accounted) ql(no effect of embedment)
3.	Allowable bearing stress given in the "Section on the design" of spread foundations.	Gravel layer Sand layer
4.	Bearing stress of cast-in-situ pile stipulated in "Section on the design of pile founation."	Sand layer and sand gravel with $30 \leq N < 40$ Sand gravel with $40 \leq N < 50$ Sand gravel with $N \geq 50$

1. Considering the fact that the minimum value of the plate bearing test result (qe) corresponds with the bearing stress for the design of shallow foundation, as seen in No. 2 & 3 in table 2, it is proper to assume that the maximum value of the bearing stress for the caisson foundation falls within the range of 150 - 200 t/m^2 which is the minimum value for qd'.
2. As for the effect of embedment on the point bearing stress, 4.8 Df(t/m^3) with the average unit weight of soil at the area of embedment taken at $\gamma = 0.8$ t/m^3, for the term $18\gamma Df$ adopted in the explanatory note of item 4.9 in this section, is taken as the bearing stress under the effect of embedment.
3. For pneumatic caisson, the minimum value of plate loading test result, 30 t/m^3, in added to the above 4.8 Dft/m^3.
4. Where the value of Df is small, assumption of the bearing stress is made as the extension of the values for spread foundations.
5. Bearing stress of open caisson upon sandy ground should be coordinated with that of 100 t/m^2 in the case of cast-in-situ pile foundation.

Based on the above considerations, the upper limit of bearing stress for the design of caisson foundations can be established as illustrated in Fig. 4-8.



Note) It is advisable that classification of soil between the sand gravel and sand layers should be carefully made based on cautious and detailed soil investigation result. Also, sand layer mixed with gravel ingradients should desirably be classified as sand layer.

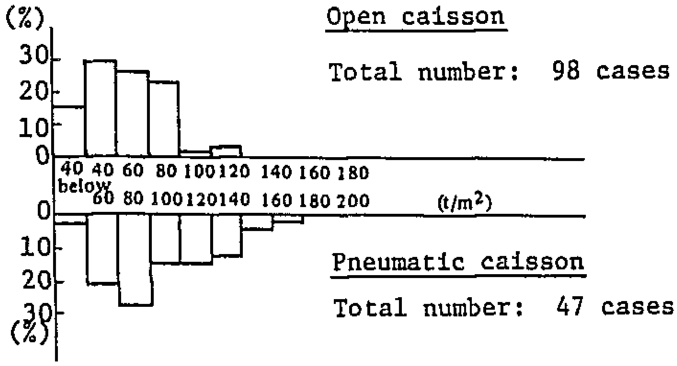
Fig. 4-8 maximum value of bearing stress for design of caisson foundation on sandy soil (under normal condition).

Fig. 4-8 is proposed based on rather bold assumptions. However as an engineering judgement, it is considerably safe to make use of such a value as a bearing stress for design purpose. In the execution of caisson foundation works, however, the actual bearing stress should be determined based on individual judgement of the actual circumstances without adhering to this proposed value, in cases where special measures are adopted for proper and cautious control of work execution and confirmation of the bearing strength.

Fig. 4-9 and 4-10 present the distribution of design value of base reaction stress for caisson foundation of road bridges constructed after the publication of the "Section on design of caisson foundation" in March, 1970, arranged based on the following conditions.

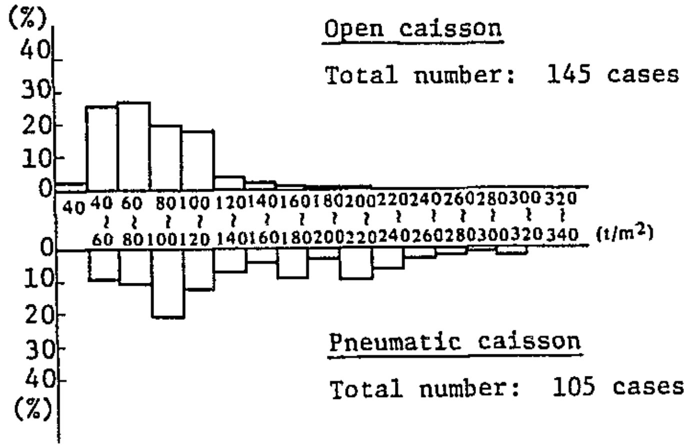
1. All collected cases have sand gravel, sand or sandy subsoil, with the majority being sand gravel of $N \geqq 40$

2. Not all the data are complete in both cases of reaction, i.e. under normal condition and during earthquake. In many cases only either of these is connected.



Note) All cases are of sandy subsoil (sand gravel and sand)

Fig. 4-9 Frequency distribution (Histogram) of maximum bottom reaction stress under normal condition.



Note) All cases are of sandy subsoil (sand gravel and sand)

Fig. 4-10 Frequency distribution (Histogram) of maximum bottom reaction stress during earthquake.

3. For the maximum base reaction stress, whichever is larger

between the stress in the longitudinal direction and that in the cross sectional direction is taken, for both the cases under normal condition and during earthquake.

4. The largest bottom area among the cases are as follows:

   Max. of circular section --- 13 m (133 m^2): Open caisson
   Max. of oval section --- 24 x 8 m (178 m^2): Pneumatic caisson

5. The largest depths of embedment are 57 m for open caisson and 40 m for pneumatic caisson respectively.

Although the samples of this data vary considerably between each other in soil conditions, shape and size of caisson, and therefore cannot be compared on the same basis, some general tendency can be observed as in the following.

Between open caissons and pneumatic caissons, there is an obvious difference in the maximum value of base reaction stress, with former being the smaller. As far as the values under normal condition are concerned, 90% of the open caissons are under 100 t/m^2 and the same proportion of the pneumatic caissons are below 140 t/m^2. Similar tendency exists in the case of the value during earthquake, with 90% of the open caisson under 120 t/m^2 and the same percentage of pneumatic caissons below 240 t/m^2.

Judging from the above data, the maximum value of base reaction stress for caisson foundations of road bridges on sandy bearing soil can be said to be as follows in the past.

Open caisson: below 100 t/m^3
Pneumatic caisson: below 150 t/m^3

Examples of Plate Bearing Test

No.	N value of bearing soil	Caisson: Base area (m)	Caisson: Embed-ment Df (m)	Design value: During earth-quake (t/m^2)	Design value: Under normal condition (kg/cm^3)	Plate bearing test data: Size of plate (cm)	Max. load: load (t/m^2)	Max. load: settle-ment (cm)	Ultimate result	Corrected ultimate bearing stress: q (t/m^2)	rDfNg Amount of cor. (t/m^2)	qd'= qi+γD fNg (t/m^2)	K= 1/2KS (kg/cm^3)	Kv (kg/cm^3)	Remarks
1	Coarse sand above 50	Oval 7.2x20.2	38	223	3.0	33	260	1.52	C	390	547	937	49	2.8	O.K.
2	Fine sand above 50	Oval 8.2x24.2	36	232	2.7	"	310	2.53	A	310	518	828	44	2.2	"
3	Coarse sand above 50	Oval 10.2x18.2	32	244	2.9	"	260	2.68	B	225	460	685	52	2.9	"
4.	Coarse sand above 50	Oval 10.2x17.2	32	240	3.0	"	280	3.08	"	375	460	835	46	2,7	"
5	Coarse sand above 50	Oval 10.2x17.2	32	263	3.0	"	280	1.84	"	300	460	760	38	2.2	"
6	Coarse sand above 50	Oval 8.2x24.2	34	209	2.7	"	300	2.31	"	255	489	744	46	2.3	"
7	Fine sand above 50	Oval 8.2x17.2	32	219	8.1	"	260	2.05	"	270	460	780	44	2.7	"
8	Fine sand above 50	Oval 8.2x17.2	34	224	3.1	"	280	1.15	"	360	483	849	34	2.1	"
9	Coarse sand above 50	Oval 8.2x22.2	33	179	2.8	"	260	2.15	"	330	475	805	40	2.1	"
10	Coarse sand above 50	Oval 7.2x22.2	34	164	2.9	"	240	1.70	"	255	489	744	54	3.1	"
11	Fine sand wi h gravel mixture	Oval 8x12	20	291	4.5	"	300	2.64	"	210	288	493	49	4.4	Inadequate bearing strength
12	Fine sand with gravel mixture	Oval 8x12	20	311	4.5	"	280	2.80	"	150	288	488	30	2.7	"
13.	Sand gravel	Oval 8x12	22	315	4.5	"	280	2.61	A	280	316	596	41	3.5	"
14.	Fine sand above 50	Circular D=3.2	27	301	3.9	"	190	5.14	"	190	388	578	7	0.6	Amount of settlement to be stadied
15	Fine sand Over 50	Circular D=3.2	30	192	3.9	33	150	3.46	B	180	431	612	10	0.8	"
16	Fine sand Over 40	Oval 7.5x10	40	75	3.2	45	345	1.00	"	420	576	806	97	10.3	O.K.
17	Fine sand 40	Oval 7.5x10	40	91	3.2	"	320	0.91	"	390	576	966	78	8.2	"
18	Sand gravel 50	Oval 10x14	34	66	3.1	33	222	1.22	"	215	489	704	125	7.9	"
19	Sand gravel "	Rectangular 10.2x22.2	20	160	2.5	75	470	6.8	A	470	288	753	66	5.1	"
20	Sand gravel	"	20	215	2.5	33	425	9.0	B	450	288	788	16	0.7	Settlement to be studied
21	Sand gravel	"	20	215	2.5	75	420	9.5	A	420	288	708	40	3.1	O.K.
22	Sand gravel	"	20	200	2.5	33	520	8.5	"	520	288	808	16	0.7	Settlement to be studied
23	Not identi-fied	Rectangular 7.7x15.2	30	165	Not id.	30	Not id.	Not id.	B	475	432	907	67	3.8	O.K.
24	"	"	30	117	"	"	"	"	"	250	432	682	33	3.6	"
25	"	Circular D=6	21	133	"	Not id.	"	"	"	105	302	407	27	2.7	"

Note: The cases of which qd' satisfied the safety factor and which have satisfactory settlement characteristics (Kv) are marked with "OK" in the "Remarks" collumn.

(7) Calculation of the passive earth pressure

The coefficient of earthpressure will be taken from the following

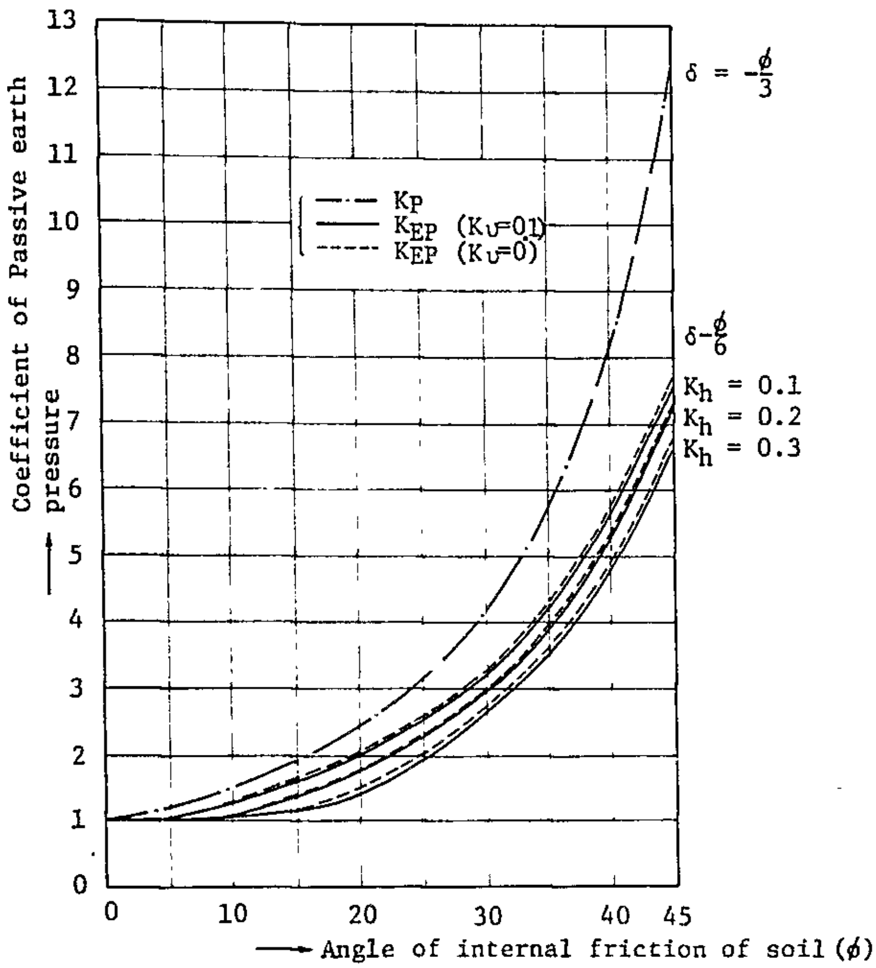


Fig. 4-11

* Estimation of the required depth of embedment for cassion P3.

As penetration test for the soil at the site of cassion P3 has been terminated before sufficient N value of the bore hole has been obtained, the required depth of embedment is established by estimation based on studies of bore hole data for soil strata and geology of the surrounding areas. This leads to the conclusion that by penetrating to the depth of approximately 8.0 - 9.0 m below the upper the level of the sand layer, an N value of 40 could be obtained, and that the depth of embedment should be ℓ = 400 m.

Chapter 5 DESIGN OF THE 3-SPAN CONTINUOUS BOX GIRDER PC BRIDGE

1. General

(1) The structure is composed of a 3-span continuous box girder with a hinge, having intermediate supports rigidly bound to the piers.

(2) For the execution of concrete works, the cantilevering cast-in-situ method with the use of a working wagon (Dywidag method) is adopted.

In the design, calculations are made according to the process of execution.

(3) PC steel bars to be used including the main bars, diagonal bars (shearing resistant bars) and the slab reinforcement bars are all to be SBPR 95/120, type B-2, ϕ32 mm size.

(4) The strength of the concrete for the main girder is to be $\sigma ck = 350\ kg/cm^2$.

(5) After deciding the length of the span and the type of the structure, the superstructure is designed according to the following process. (Fig. 5-1)

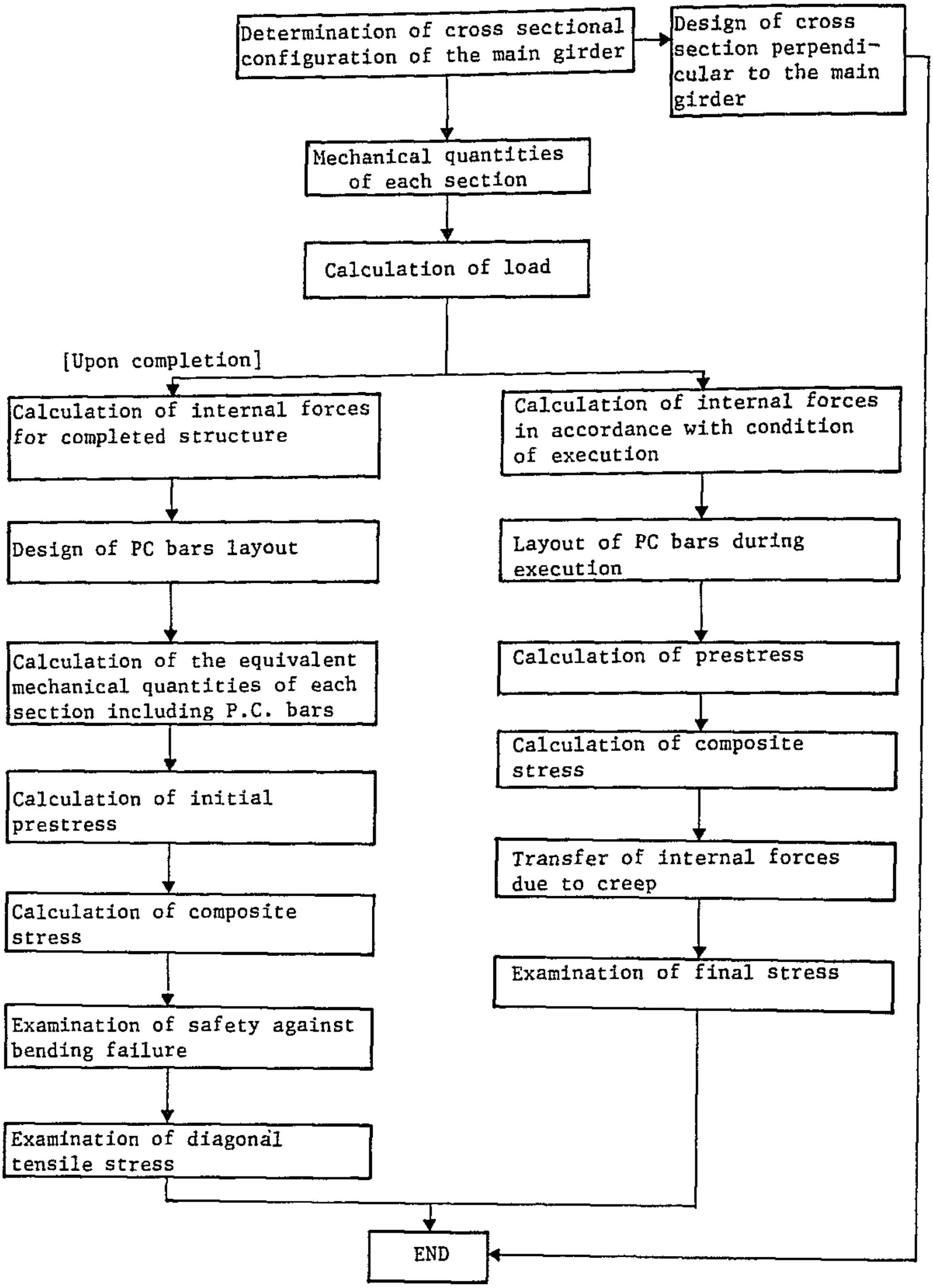

Determination of cross sectional configuration of the main girder
Design of cross section perpendicular to the main girder
Mechanical quantities of each section
Calculation of load
[Upon completion]
Calculation of internal forces for completed structure
Design of PC bars layout
Calculation of the equivalent mechanical quantities of each section including P.C. bars
Calculation of initial prestress
Calculation of composite stress
Examination of safety against bending failure
Examination of diagonal tensile stress
Calculation of internal forces in accordance with condition of execution
Layout of PC bars during execution
Calculation of prestress
Calculation of composite stress
Transfer of internal forces due to creep
Examination of final stress
END

Fig. 5-1

2. Design Condition

(1) Type and load

Bridge type:	Prestress concrete bridge
Bridge length:	240.0 m
Span:	69.5 m + 100.0 m + 69.5 m
Type of structure:	3-span continuous PC box girder with central hinge, Dywidag method
Road construction standard:	
Width:	1.500 m + 8.000 m + 1.500 m
Live load:	TL-20
Impact coefficient:	Main girder $i = \frac{10}{25 + \ell}$
	Slab $i = \frac{20}{50 + \ell}$
Design seismic coefficient:	Horizontal seismic coefficient: Kh = 0.12
Temperature change:	$\pm 15°C$

(2) Allowable Stress for Main Materials

a) Concrete

Design standard strength: Main girder $\sigma ck = 350\ kg/m^3$

The values of allowable compressive stress for concrete are as given in following table.

Allowable compressive stress of concrete (kg/cm^2)

<table>
<tr><th colspan="3">Type of stress \ Design standard strength for concrete</th><th>300</th><th>350</th><th>400</th></tr>
<tr><td rowspan="3">Immediately after prestressing</td><td rowspan="2">Bending compression stress</td><td>(1) For rectangular section</td><td>150</td><td>170</td><td>190</td></tr>
<tr><td>(2) For T & Box section</td><td>140</td><td>160</td><td>180</td></tr>
<tr><td colspan="2">(3) Axial compressive stress</td><td>110</td><td>127.5</td><td>145</td></tr>
<tr><td rowspan="3">Others</td><td rowspan="2">Bending compression stress</td><td>(4) For rectangular section</td><td>120</td><td>135</td><td>150</td></tr>
<tr><td>(5) For T & Box section</td><td>110</td><td>125</td><td>140</td></tr>
<tr><td colspan="2">(6) Azial compressive stress</td><td>85</td><td>97.5</td><td>110</td></tr>
</table>

<table>
<tr><th colspan="3">Type of stress \ Design standard strength for concrete</th><th>300</th><th>350</th><th>400</th></tr>
<tr><td rowspan="4">Bending tensile stress</td><td colspan="2">(1) Immediately after pre stressing</td><td>12</td><td>13.5</td><td>15</td></tr>
<tr><td colspan="2">(2) Life load & main loads other than impact load</td><td>0</td><td>0</td><td>0</td></tr>
<tr><td rowspan="2">Main load and equivalent special loads</td><td>(3) For joints of slabs, or joints of blocks in the case of block method being used</td><td>0</td><td>0</td><td>0</td></tr>
<tr><td>(4) In other cases</td><td>12</td><td>13.5</td><td>15</td></tr>
<tr><td colspan="3">(5) Axile tensile stress</td><td>0</td><td>0</td><td>0</td></tr>
</table>

<table>
<tr><th colspan="2">Type of stress \ Design standard strength for concrete</th><th>300</th><th>350</th><th>400</th></tr>
<tr><td colspan="2">(1) Shearing stress</td><td>4.8</td><td>5</td><td>5.5</td></tr>
<tr><td rowspan="2">Diagonal tensile stress</td><td>(2) In cases where shearing force alone or torsion moment alone is to be considered</td><td>8</td><td>9</td><td>10</td></tr>
<tr><td>(3) In cases where both shearing force and torsion moment are to be considered</td><td>11</td><td>12</td><td>13</td></tr>
</table>

Allowable bond stress of concrete (kg/cm^2)

Type of reinforceing bar \ Design standard strength for concrete	300	350	400
(1) Ordinary round bar	9	9.5	10
(2) Deformed bar	18	19	20

b) PC steel bar (Type B No. 2, SBPR 95/120 ϕ32)

Formula for allowable tensile stress of PC steel materials

Condition of stress	Allowable tensile stress	
(1) At initial loading of tensile force	0.80 σ_{pu} or 0.90 σ_{py} whichever is smaller	σpu: Tensile strength of PC steel σpy: Yielding stress of PC steel
(2) Immediately after prestressing in the case of post-tension method	0.7 σ_{pu} or 0.85 σ_{py} whichever is smaller	
(3) Other cases	0.6 σ_{pu} or 0.75 σ_{py} whichever is smaller	

Allowable tensile stress of PC bar

		Condition of stress	(1)	(2)	(3)
Steel bar	Round bar Type A	No. 1 SBPR 80/95	72.0	66.5	57.0
		No. 2 SBPR 80/105	72.0	68.5	60.0
	Round bar Type B	No. 1 SBPR 95/110	85.5	77.0	65.0
		No. 2 SBPR 95/120	85.5	80.7	71.2

PC steel bars to be used for this bridge

Main bar: SBPR 95/120 ϕ32 mm

Shearing resistant bar: SBPR 95/120 ϕ32 mm

Tensile Strength:

Main bar: $\sigma pu = 120\ kg/mm^2$

Shearing resistant bar: $\sigma pu = 120\ kg/mm^2$

Yielding Stress:

Main bar: $\sigma py = 95\ kg/mm^2$

Shearing resistant bar: $\sigma py = 95\ kg/mm^2$

Allowable Tensile Force (per piece):

Main bar: $Pt = 63.1$ t/piece

Shearing resistant bar: $Pt = 63.1$ t/piece

Under Design Load:

Main bar $Pe = 56.2$ t/piece

Shearing resistant bar: $Pe = 56.2$ t/piece

c) Reinforcing bar (SD 30)

Allowable stress of reinforcing bars of 32 mm diameter or less are as given in followint table.

Allowable stress of reinforcing bar (kg/cm^2)

Type of stress and member / Type of steel bar			SR 24 SD 24	SD 30	SD 35
Tensile stress	Cases where impaction and seismic force taken into consideration	(1)	1,400	1,800	1,800
		(2) Slab bridge of which slab and span are not longer than 10 m	1,400	1,400	1,400
		(3) Member under water or below ground-water table	1,400	1,600	1,600
	(4) Basic value of allowable stress for cases where the effect of impaction and seismic loads are taken into account, in considering the composition of loads		1,400	1,600	2,000
	(5) For determining the lap joint or anchorage length of the steel bar		1,400	1,800	2,000
(6) Compressive stress			1,400	1,800	2,000

(3) Combination of loads under ultimate limit state

1.3 x (Dead load) + 2.5 x (Live load + Impaction)

1.0 x (Dead load) + 2.5 x (Live load + Impaction)

1.7 x (Dead load + Live load + Impaction)

1.8 x (Dead load + Effect of seismic force)

1.0 x (Dead load) + 1.3 x (Effect of seismic force)

(4) Creep and drying shrinkage of concrete

Creep coefficient for concrete:

Age of concrete under continuous load (Days)		4 - 7	14	28	90	365
Creep coefficient	High early strength portland cement	2.6	2.3	2.0	1.7	1.2
	Normal portland cement	2.8	2.5	2.2	1.9	1.4

Rate of drying shrinkage of concrete

Age of concrete undergoing prestressing (Days)	4 - 7	28	90	365
Rate of drying shrinkage	$20x10^{-5}$	$18x10^{-4}$	$16x10^{-4}$	$12x10^{-4}$

(5) Relaxation of PC steel materials

a) In calculating the diminishing amount of the prestress the figures given in following table are the standard design values for the rate of relaxation of the PC steel material.

However, in calculating the rate of relaxation for PC steel material receiving the effects of high temperatures, 2% is to be added to the figures given in the table.

Design rate of relaxation for PC steel material

Type of PC steel material	Rate of relaxation (%)
PC steel wire, PC strand	5
PC steel bar	3

(6) Young's modulus

Young's modulus of concrete

(kg/cm^2)

Design standard stressing	210	240	270	300	3.50	400
Young's modulus	$2.6x10^4$	$2.7x10^4$	$2.85x10^4$	$3.0x10^4$	3.25	3.5x10

Young modulus of steel

(kg/cm^2)

Type of steel material	Young modulus
Reinforcing bar	2.1×10^6
PC steel wire, PC strand PC steel bar	2.0×10^6

(7) Additional value of allowable stress according to type of load

3.3.4 Additional value of allowable tensile stress for concrete

The allowable tensile strength of concrete where secondary load and equivalent special loads are taken into consideration is as given in the following table. The allowable stress during errection with wind load and the effect of earthquake taken into account is separately provided taking the conditions and stractural system of errection, irregarding the stipulations in this table.

(kg/cm^2)

Combination of Load \ Design standard strength	300	350	400
(1) Main load (P) + Special load equivalent to main load + effect of temperature change (Γ)	17	18.5	20
(2) Main load (P) + Special load equivalent to main load + Wind load (w)	22	23.5	25
(3) Main load (P) + Special load equivalent to main load + effect of temperature change (Γ) + wind load (w)	22	23.5	25
(4) Main load (P) + Special load equivalent to main load + Braking load (BK)	22	23.5	25
(5) Main load (P) + Special load equivalent to main load + Collision load (CO)	-	-	-
(6) Live load and main loads other than impact load + Effect of earthquake (EQ)	-	-	-
(7) Live load and main loads other than impact load + Effect of earthquake (EQ) + Effect of temperature change (Γ)	-	-	-
(8) Wind load (W) only	20	215.	23
(9) Loads during errection (ER)	22	23.5	25

3. Determination of the Sectional Form of the Main Girder

(1) Determination of the shape of the box rigid frame

The shape of a box rigid frame is determined based on consider-ration of the balance of bending moments.

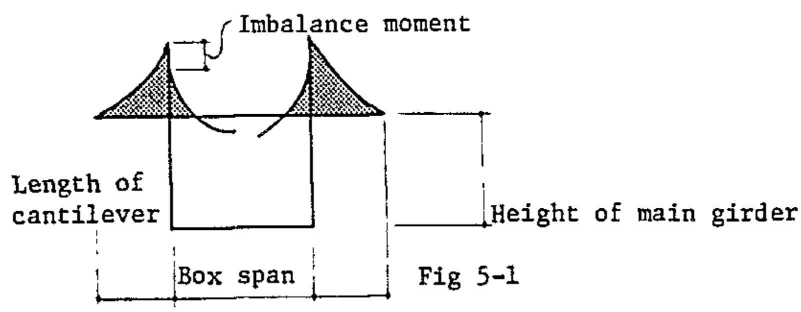


Fig 5-1

Conditions for the design of the section

Height of the main girder will be studied based on the average height.

H = 4.210 m	Thickness of top slab	t = 300
	Thickness of bottom slab	t = 400
	Thickness of web	t = 485

For the width of the box frame selection will be made from three alternatives: 6.00 m, 6.50 m, 7.00 m, based on comparative study.

Case - 1 $B = 6^{m}.000$

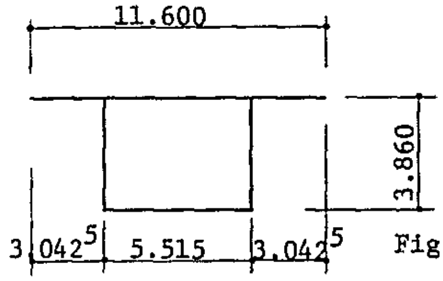


Fig 5-2

Case - 2 $B = 6^{m}.500$

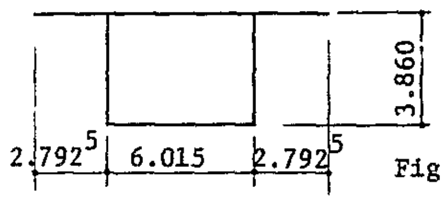


Fig 5-3

Case - 3 $B = 7^{m}.000$

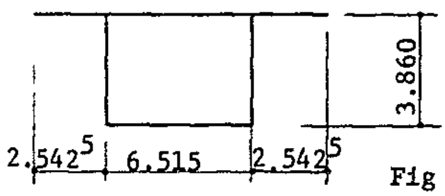


Fig 5-4

Bending moment at the joint of the web of the upper slab

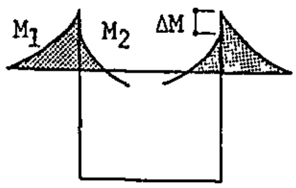


Fig. 5.5

			M1	M2	M
Case - 1	1	Dead load	-4.051	-2.657	
B = 6.0	2	Static load	-4.980	-1.090	
		1 + 2	-9.031	-3.747	
	3	Live load	-6.958	-7.524	
		1 + 2 + 3	-15.989	-11.271	-4.718
Case - 2	1	Dead load	-3.413	-2.948	
B = 6.5	2	Static load	-4.433	-1.092	
		1 + 2	-7.846	-4.040	
	3	Live load	-5.611	-8.256	
		1 + 2 + 3	-13.457	-12.296	-1.161
	1	Dead load	-2.829	-3.292	
Case - 3	2	Static load	-3.900	-3.779	
B = 7.0		1 + 2	-6.729	-7.071	
	3	Live load	-1.250	-8.986	
		1 + 2 + 3	-7.979	-16.057	8.078

From the above analysis it is clear that bending moment at the joint of the web is closest to balance under design load (dead load + live load) in case - 2, which will have the most desirable section, and will therefore, be selected for the detail design.

(2) Form of the soffit curve of the girder

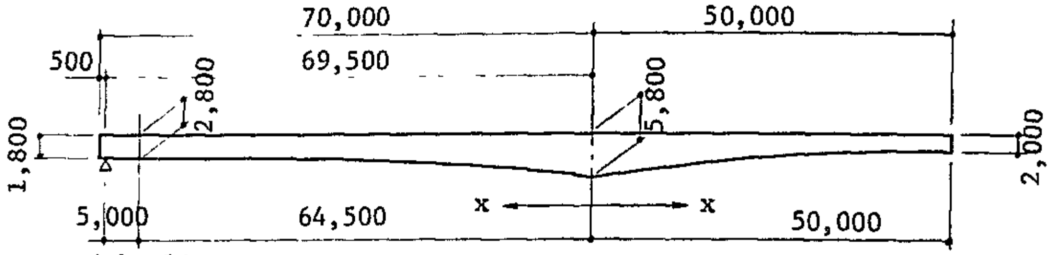


Fig. 5-6

In this design, a sine curve will be used.

Side Span: $Hx = Hp - (Hp - Ho) \sin (\frac{Rx}{2\ell})$ (180°)

$$Hx = 5.8 - (5.8 - 2.8) \sin (\frac{180° \times x}{2 \times 64.5})$$

Central span:

$$Hx = 5.8 - (5.8 - 2.5) \sin (\frac{180° \ x}{2 \times 50.0})$$

The followings are forms of curve used nowadays in the design of bridge girder.

1) Since curve $Y = a \sin \frac{\pi}{\ell} X$
2) $Y = ax^{1.5}$
3) Parabolic curve $Y = ax^2$
4) Dubic curve $Y = ax^3$

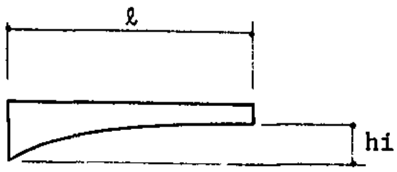


Fig. 5-7

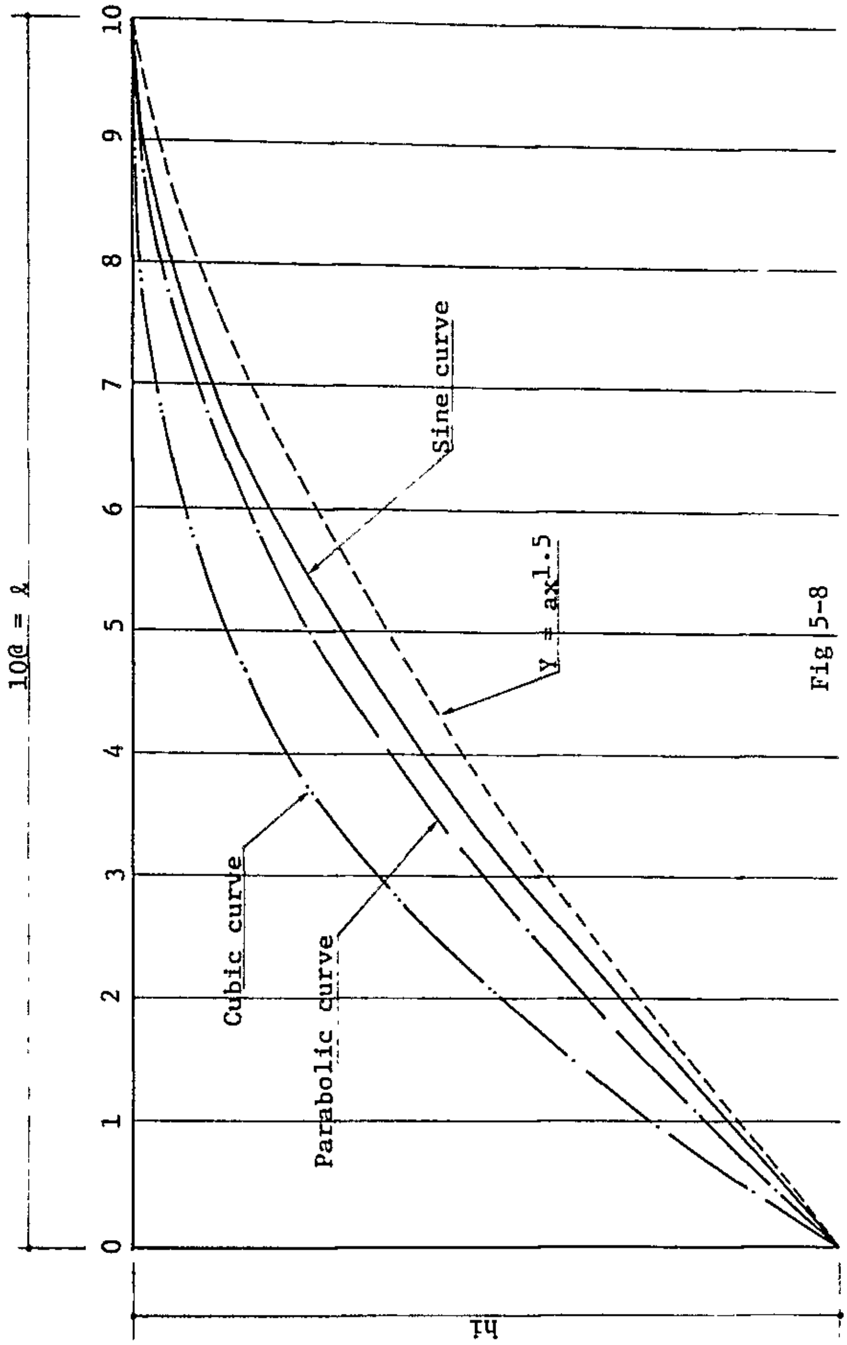
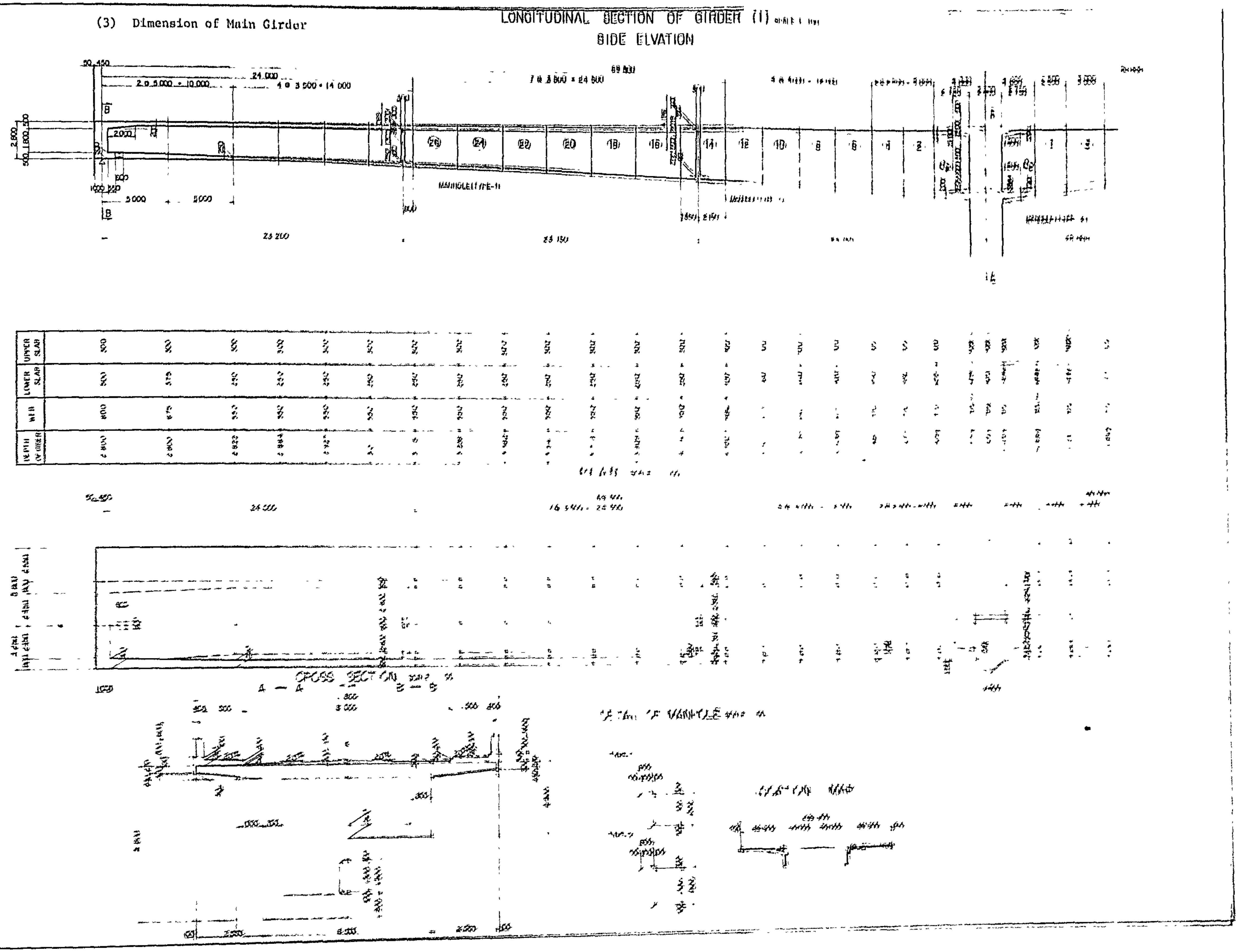


Fig 5-8

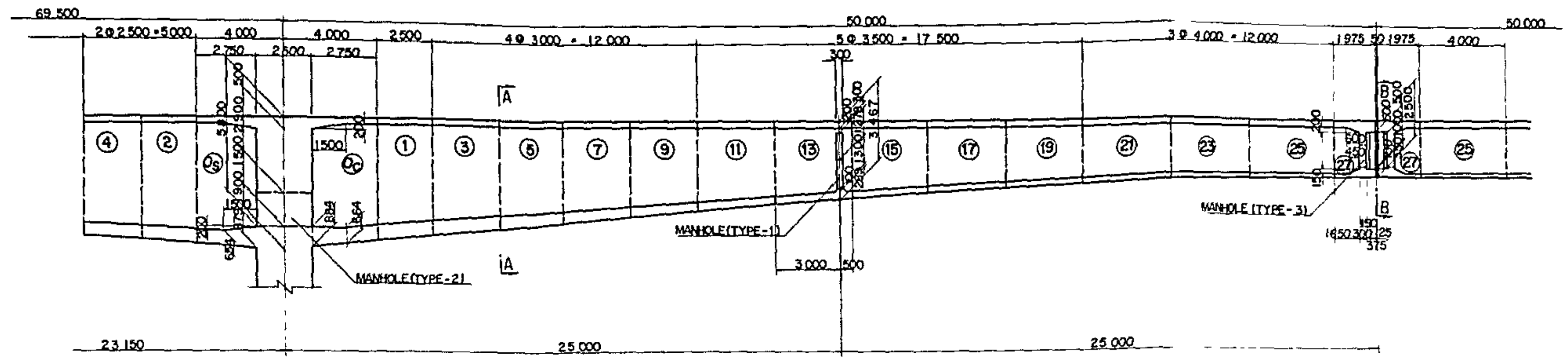
Sine curve is the most common form used in continuous girder with central hinge

(3) Dimension of Main Girder



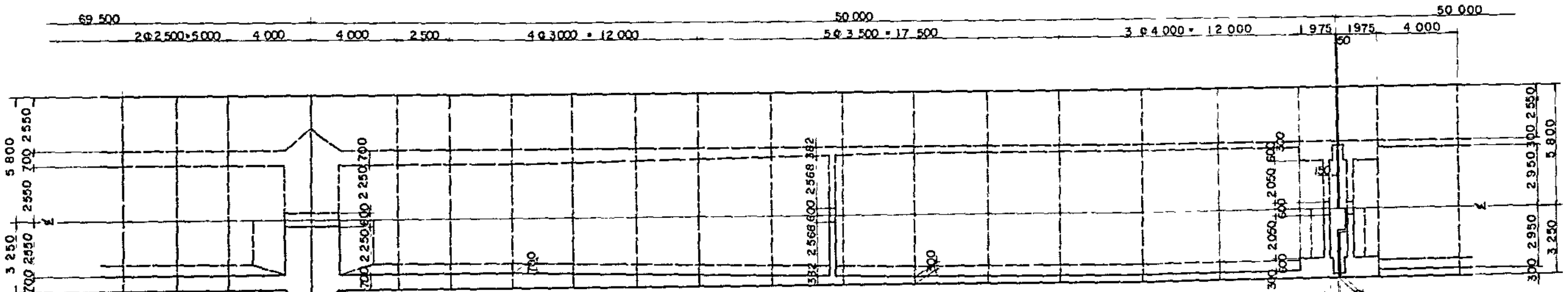
LONGITUDINAL SECTION OF GIRDER (2) SCALE 1 100

SIDE ELEVATION



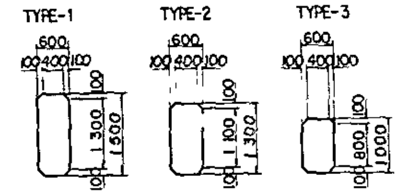
UPPER SLAB	300	300	300	500	500	500	300	300	300	300	300	300	300	300	300	300	300	300	300	300	300
LOWER SLAB	550	592	633	879	900	884	648	616	577	539	500	433	356	278	200	200	200	200	200	200	200
WEB	700	700	700	700	700	700	700	700	700	638	577	515	444	372	300	300	300	300	300	300	900
DEPTH OF GIRDER	5 147	5 327	5 508	5 709	5 800	5 670	5 386	5 131	4 830	4 537	4 256	3 988	3 697	3 430	3 192	2 986	2 814	2 662	2 558	2 507	2 500

PLAN SCALE 1 100

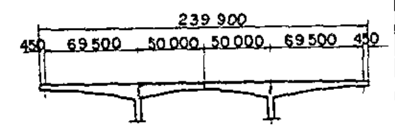


DETAIL OF MANHOLE SCALE 1 50

TYPE-1 TYPE-2 TYPE-3

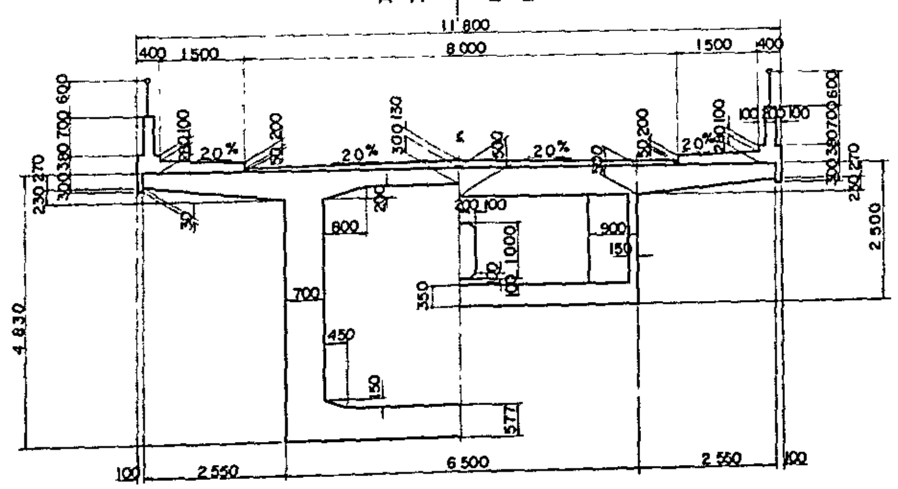


LOCATION MAP

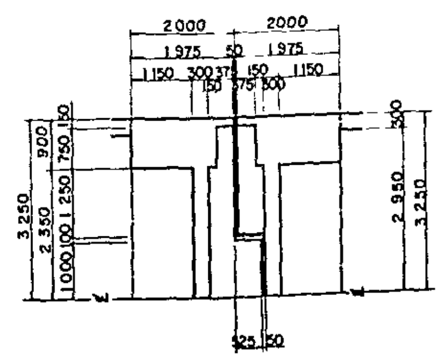


CROSS SECTION SCALE 1 50

A–A B–B



DETAIL OF CENTER HINGE SCALE 1 50



4. Structural Design of Longitudinal Sections

(1) Design and Performance of Section

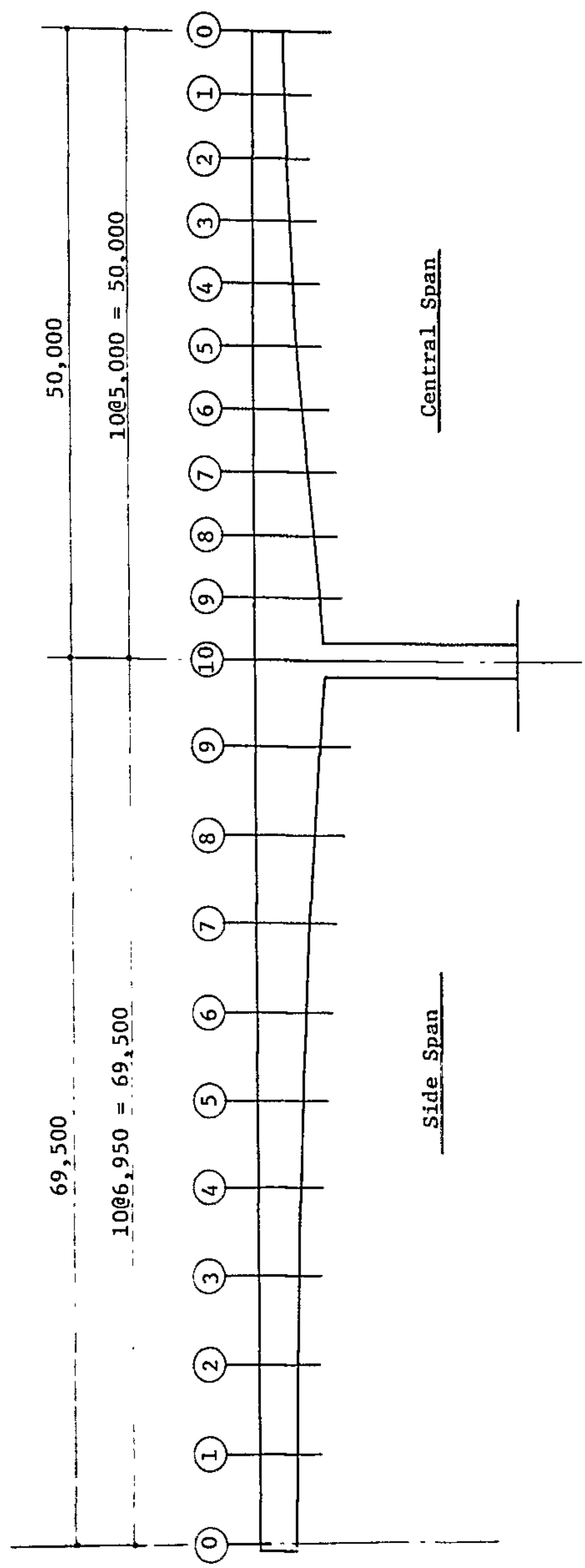


Fig. 5-9

Side span section

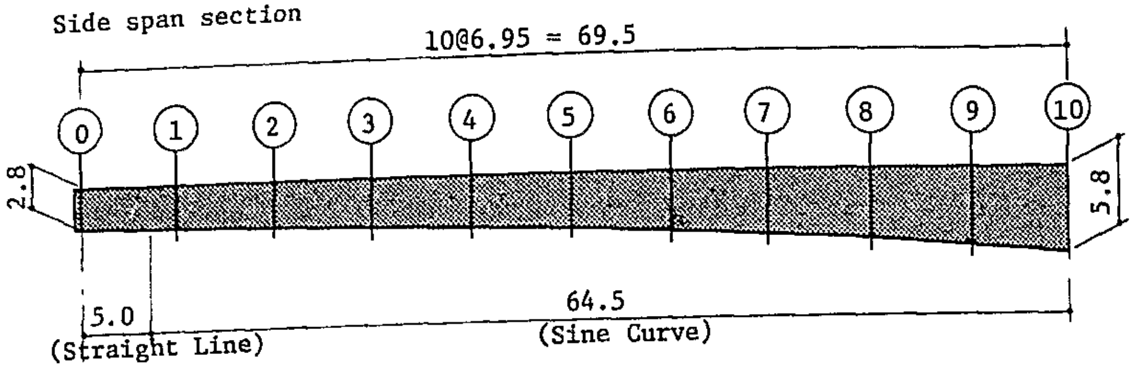


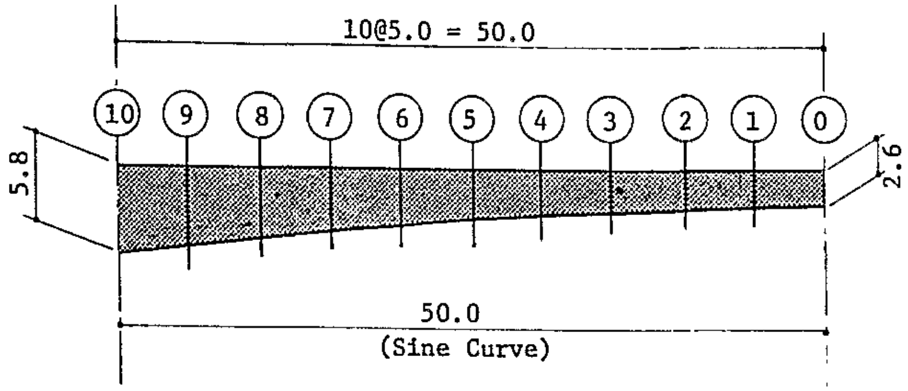
Fig. 5-10

CANTILEVER NO.1

NO	Height of girder H	Thickness of web TW	Thickness of upper slab TU	Thickness of lower slab TL	HH
0	2.800	0.800	0.300	0.500	0.000
1	2.803	0.626	0.300	0.326	0.000
2	2.870	0.550	0.300	0.250	0.000
3	3.021	0.550	0.300	0.250	0.000
4	3.251	0.550	0.300	0.250	0.000
5	3.554	0.550	0.300	0.250	0.000
6	3.921	0.550	0.300	0.250	0.000
7	4.341	0.585	0.300	0.323	0.000
8	4.804	0.653	0.300	0.462	0.000
9	5.295	0.700	0.300	0.584	0.000
10	5.800	0.700	0.300	0.700	0.000

NO	Sectional area A	Centroid location YU	YL	Moment of inertia of area I	Section modulus ZU	ZL
0	10.5910	1.2595	1.5405	11.4014	9.0521	7.4013
1	8.9885	1.1448	1.6586	9.9410	8.6839	5.9935
2	8.3182	1.0941	1.7761	9.5145	8.6959	5.3571
3	8.4838	1.1554	1.8653	10.7837	9.3331	5.7812
4	8.7368	1.2501	2.0006	12.9028	10.3216	6.4494
5	9.0699	1.3764	2.1771	16.0363	11.6505	7.3659
6	9.4736	1.5320	2.3886	20.3791	13.3026	8.5318
7	10.5933	1.8108	2.5305	28.5204	15.7503	11.2705
8	12.4191	2.1745	2.6293	41.1496	18.9241	15.6505
9	14.1128	2.5168	2.7778	56.1527	22.3110	20.2145
10	15.4110	2.8566	2.9434	73.3764	25.6868	24.9290

NO	Geometric moment of area QN	Q1	Q2	Thickness of web QB	Length of section DL	Curve of girder soffit) SL
0	5.1269	4.6654	4.4926	0.8000	0.0000	0.000000
1	4.3697	4.1093	3.4942	0.6263	6.9500	0.003468
2	4.0635	3.8693	3.0220	0.5500	6.9500	0.015712
3	4.3687	4.1325	3.1878	0.5500	6.9500	0.027506
4	4.8483	4.5388	3.4392	0.5500	6.9500	0.038515
5	5.5039	5.0814	3.7669	0.5500	6.9500	0.049422
6	6.3348	5.7491	4.1598	0.5500	6.9500	0.056946
7	7.9718	6.9661	5.4939	0.5853	6.9500	0.063842
8	10.4102	8.5805	7.7546	0.6526	6.9500	0.068914
9	12.9518	10.1046	10.0282	0.7000	6.9500	0.072016
10	15.4712	11.5837	12.4035	0.7000	6.9500	0.073060



CANTILEVER NO.2

NO	H	TW	TU	TL	HH
0	2.500	0.300	0.300	0.200	0.000
1	2.541	0.300	0.300	0.200	0.000
2	2.662	0.300	0.300	0.200	0.000
3	2.860	0.300	0.300	0.200	0.000
4	3.130	0.300	0.300	0.200	0.000
5	3.467	0.382	0.300	0.289	0.000
6	3.860	0.485	0.300	0.400	0.000
7	4.302	0.587	0.300	0.506	0.000
8	4.780	0.690	0.300	0.571	0.000
9	5.284	0.700	0.300	0.635	0.000
10	5.800	0.700	0.300	0.700	0.000

NO	A	YU	YL	I	ZU	ZL
0	6.6410	0.8391	1.6609	5.7727	6.8798	3.4756
1	6.6654	0.8528	1.6878	5.9992	7.0352	3.5548
2	6.7379	0.8940	1.7675	6.7048	7.4996	3.7934
3	6.8568	0.9624	1.8972	7.9568	8.2672	4.1939
4	7.0191	1.0575	2.0727	9.8640	9.3273	4.7590
5	8.2176	1.3402	2.1264	14.9894	11.1847	7.0493
6	9.8041	1.6571	2.2032	22.2954	13.4544	10.1196
7	11.5378	1.9697	2.3321	31.7882	16.1383	13.6307
8	13.2451	2.2545	2.5257	43.2921	19.2467	17.1801
9	14.3592	2.5506	2.7331	57.0584	22.3703	20.8766
10	15.4110	2.8566	2.9434	73.3764	25.6868	24.9290

NO	QN	Q1	Q2	QB	DL	SL
0	2.7647	2.7303	2.2492	0.3000	0.0000	0.000000
1	2.8253	2.7879	2.2884	0.3000	5.0000	0.016218
2	3.0073	2.9607	2.4045	0.3000	5.0000	0.032037
3	3.3118	3.2476	2.5936	0.3000	5.0000	0.047066
4	3.7397	3.6464	2.8493	0.3000	5.0000	0.060937
5	5.1322	4.8625	4.0443	0.3921	5.0000	0.073308
6	6.9024	6.2535	5.5779	0.4946	5.0000	0.083873
7	8.9204	7.6520	7.2717	0.5872	5.0000	0.092373
8	11.0783	8.9551	8.8317	0.6897	5.0000	0.098598
9	13.1954	10.2518	10.5401	0.7000	5.0000	0.102396
10	15.4712	11.5837	12.4035	0.7000	5.0000	0.103673

(2) Load Calculation

Static load:

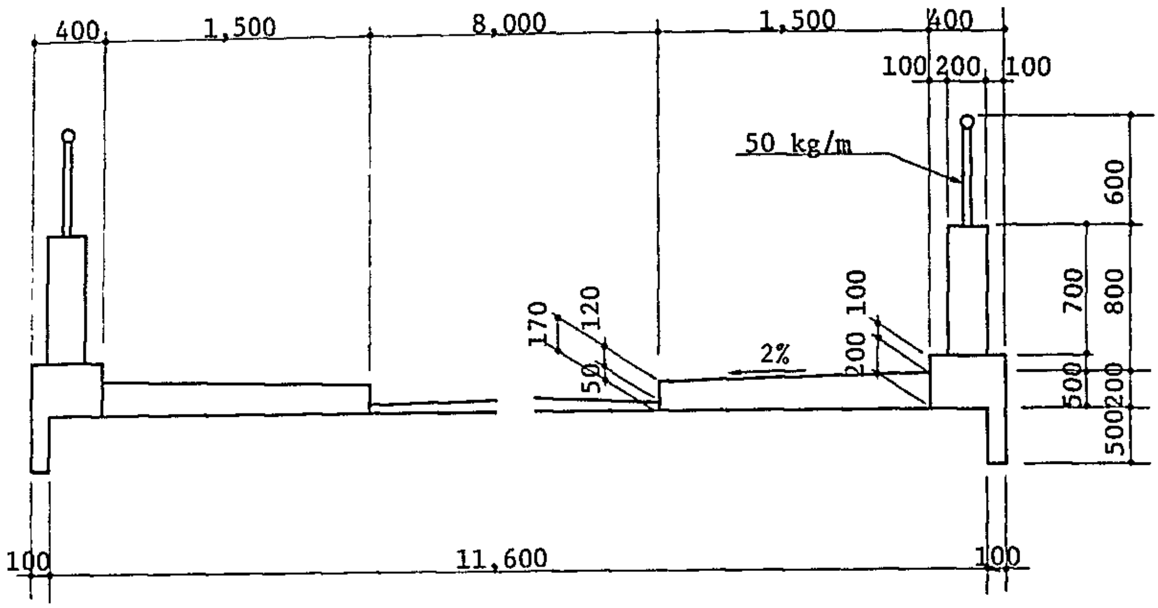


Fig. 5-12

Surfacing and railing

[(0.20 x 0.70) + (0.40 x 0.30) + (0.30 + 0.10)	
+ 1/2 (0.17 + 0.20) x 1.5] x 2.5 x 2	= 2.838 t/m
0.050 x 2	= 0.100 t/m

Pavement

1/2 (0.050 + 0.130) x 8.0 x 2.3	= 1.656 t/m
Total	= 4.594 t/m

Live load (TL - 20)

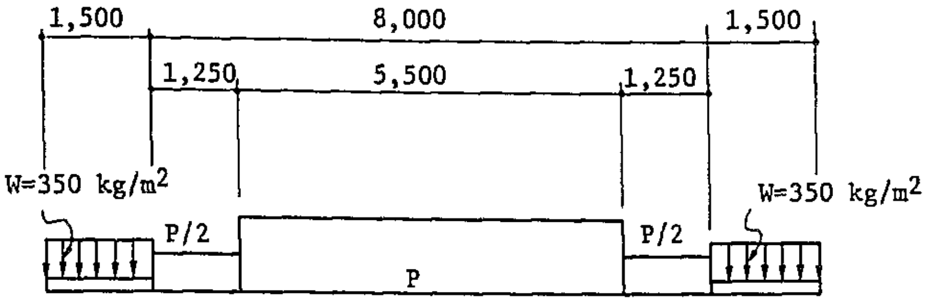


Fig. 5-13

Uniform load:

$$W = [5.5 + \frac{(8.0 - 5.5)}{2}] \times 0.350 = 2.363 \text{ t/m}$$

Linear load:

$$P\ [5.5 + \frac{(8.0 - 5.5)}{2}] \times 5.0 \qquad = 33.750 \text{ t}$$

Side walk load

$$W' = 0.350 \times 1.5 \times 2 \qquad = 1.050 \text{ t/m}$$

Impact coefficient

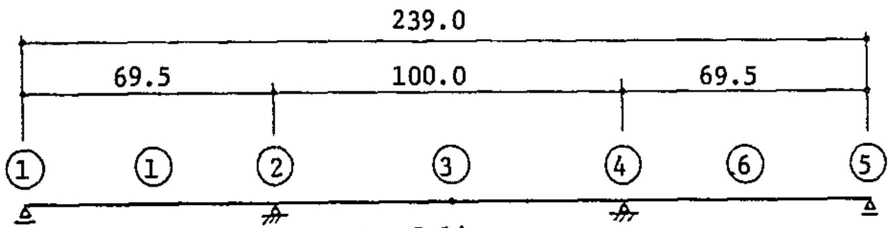


Fig. 5-14

$$i = \frac{10}{25 + L}$$

Item \ Position	1	2	3	4	5
L	69.5	84.75	100.0	84.75	69.5
Impact coefficient	0.1058	0.0911	0.0800	0.0911	0.1058

(3) Computation of internal force

The basic structural system of this bridge is indeterminate by three degree, as illustrated in the following figure, in which X_1, X_2, and X_3 indicate the statically indicate the statically indeterminate force.

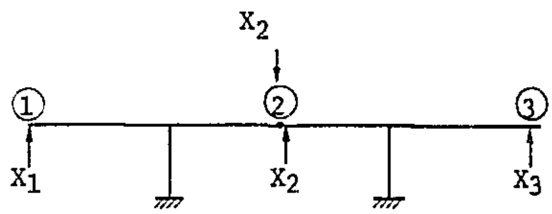


Fig. 5-15

For the basic system in the above figure, elastic equations can be established as follows.

$$\begin{aligned} \delta 11.\ X1 + \delta 12\ .\ X2 + \delta 13.\ X3 + \delta 10 &= 0 \\ \delta 21.\ X1 + \delta 22.\ X2 + \delta 23.\ X3 + \delta 20 &= 0 \qquad \text{Formulas (1)} \\ \delta 31.\ X1 + \delta 32.\ X2 + \delta 33.\ X3 + \delta 30 &= 0 \end{aligned}$$

Here, $\delta 11$, $\delta 21$, $\delta 31$ are the deflection of points 1, 2, 3 respectively when a unit load x = 1 ton is imposed at a certain point upon the bridge, and $\delta 12$, $\delta 22$, $\delta 32$ are the actual deflection of points 1, 2, 3 respectively when two points on the bridge are imposed each with a unit load P = 1 ton. From the reversal theory of Betti-Maxwell and the condition of structural symmetry, the following formulas are establishdd.

$$\delta 11 = \delta 33$$

$$\delta 12 = \delta 21 = \delta 23 = \delta 32$$

$$\delta 13 = \delta 31 = 0$$

$$\delta 30 = 0$$

Incorporating these with formulas (1), the following formulas are obtained.

$$\delta 11.\ X1 + \delta 12.\ X2 + \delta 10 = 0$$

$$\delta 12.\ X1 + \delta 22.\ X2 + \delta 12.\ X3 + \delta 20 = 0 \quad \text{Formula (2)}$$

$$\delta 12.\ X2 + \delta 11.\ X3 = 0$$

Rendering the third formula of (2) into

$$X3 = -\frac{\delta 12.\ X2}{\delta 11}$$

and substituting this into the second formulas of (2) the followings are obtained.

$$\delta 12.\ X1 + \delta 22.\ X2 + (\frac{\delta 12.\ X2}{\delta 11}) = -\delta 20$$

$$\delta 12.\ X1 + (\delta 22 - \frac{\delta 12}{\delta 11})\ x\ 2 = -\delta 20$$

Substituting the followings taken from the calculation papers.

$$\delta 11 = 4.5145 \times 10^{-3}$$

$$\delta 12 = 2.4064 \times 10^{-3}$$

$$\delta 22 = 2.898 \times 2 = 4.3795 \times 10^{-3}$$

The followings are obtained (with 10^{-3} omitted)

$$4.5145X1 + 2.4064X2 = -\delta 10 \quad (3)$$

$$2.4064X1 + (4.3795 - \frac{2.4064^2}{4.5145})X2 = -\delta 20 \quad (4)$$

Rendering formula (4) again,

$$2.4064X1 + 3.0968X2 = -\delta 20 \quad (4\text{-}1)$$

and from formulas (3) and (4-1), the following are obtained.

$$X1 = \frac{0.7771\delta 20 - \delta 10}{2.6446}$$

$$X2 = \frac{\delta 10 - 1.876\delta 20}{3.4033}$$

***BURMA DH (SPAN 69.5+100.0+69.5 PIER 19.937)

BENDING-MOMENT

	G	GS	P +	P - (M'N	Q +	Q -	P1	Q1+	Q1-	Q2
1.. 0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.. 1	2647.99	466.79	272.96	-67.52	507.51	-236.36	-31.36	203.96	-97.25	0.00
1.. 2	4191.60	711.67	375.14	-135.03	888.80	-472.72	-62.72	357.15	-194.49	0.00
1.. 3	4714.02	734.65	462.82	-202.55	1143.88	-709.08	-94.08	459.65	-291.74	0.00
1.. 4	4164.17	535.73	495.06	-270.07	1272.75	-945.44	-125.44	511.46	-388.99	0.00
1.. 5	2534.17	114.91	481.86	-337.59	1275.40	-1181.80	-156.80	512.50	-486.24	0.00
1.. 6	-172.49	-527.51	432.73	-405.10	1151.83	-1418.16	-188.16	462.95	-583.49	0.00
1.. 7	-4109.56	-1392.43	355.80	-472.62	902.05	-1654.53	-219.52	382.48	-680.73	0.00
1 . 8	-9390.71	-2479.96	256.15	-540.14	624.22	-1989.05	-250.88	250.84	-817.42	0.00
1.. 9	-16149.88	-3787.39	136.74	-607.66	460.54	-2563.94	-282.24	195.06	-1050.70	0.00
1..10	-24615.30	-5317.72	138.01	-675.17	427.51	-3395.69	-313.60	171.79	-1387.20	0.00
1..10	-25398.60	-5742.50	186.26	-911.25	576.99	-3767.04	-423.25	231.86	-1544.36	0.00
1.. 9	-19796.27	-4651.43	167.63	-820.13	529.25	-3113.19	-380.93	212.77	-1275.89	0.00
1.. 8	-15090.75	-3675.20	149.01	-729.00	497.93	-2539.57	-338.60	200.44	-1040.44	0.00
1.. 7	-11206.89	-2813.93	130.38	-637.88	477.54	-2040.67	-296.27	192.60	-835.73	0.00
1.. 6	-8043.93	-2067.30	139.49	-546.75	461.62	-1610.04	-253.95	186.60	-659.10	0.00
1.. 5	-5495.09	-1435.63	156.29	-455.63	442.76	-1240.28	-211.63	179.40	-507.52	0.00
1.. 4	-3525.86	-919.90	162.23	-364.50	412.47	-922.87	-169.30	167.49	-377.49	0.00
1.. 3	-2006.13	-516.83	154.19	-273.38	361.35	-648.46	-126.97	147.01	-265.14	0.00
1.. 2	-915.41	-229.70	127.34	-182.25	280.37	-407.97	-84.65	114.25	-166.75	0.00
1.. 1	-246.29	-57.43	77.05	-91.13	161.87	-193.77	-42.33	66.05	-79.17	0.00
1.. 0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

***BURMA DH.(SPAN 69.5+100.0+69.5 PIER 19.937)

BENDING-MOMENT

	D3	EQ	SJ	P +	P -	Q +	Q -	Q1+	Q1-
1 . 0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.. 1	0.00	-175.07	46.70	222.96	-67.52	453.26	-182.11	192.14	-75.45
1.. 2	0.00	-350.14	93.41	375.14	375.14	706.76	-290.68	254.00	-121.34
1. 3	0.00	-525.22	140.11	462.92	462.82	810.62	-375.87	325.74	-157.93
1.. 4	0.00	-700.29	186.51	495.06	495.06	810.24	-482.94	325.58	-203.14
1.. 5	0.00	-875.35	233.52	481.96	491.86	745.00	-651.41	299.37	-273.10
1.. 6	0.00	-1050.43	280.22	432.73	432.73	648.03	-914.36	260.40	-391.04
1 . 7	0 00	-1225.50	326.92	359.90	355.80	546.64	-1299.11	219.66	-597.91
1.. 8	0.00	-1400.57	373.63	256.15	256.15	463.47	-1828.30	196.24	-752.83
1 9	0.00	-1575.65	420.33	136.74	136.74	418.03	-2521.43	-167.98	-1033.62
1..10	0 00	-1750 72	467.06	139.01	0.00	427.51	-3395.69	171.70	-1387.20
1..10	0 00	2311 44	472.55	186.26	0.00	576.99	-3767.04	231.86	-1544.35
1 9	0.00	2080.30	390 30	167.63	43.04	526.83	-3110.77	211.77	-1274.90
1 . 8	0.00	1849 15	338.05	149.01	81.41	489.24	-2530.87	196.86	-1016.85
1 7	0.00	1618.01	295.79	130.38	114.00	460.50	-2023.63	185.59	-829.72
1.. 6	0.00	1386 87	253.54	139.49	139.49	436.24	-1584.66	176.16	-649.65
1 5	0.00	1155 72	211.29	156.29	156.29	411.74	-1208.75	166.43	-494.55
1.. 4	0 00	924.58	169.02	162.23	162.23	379.27	-889.68	153.83	-363.83
1 . 3	0.00	693 43	126.77	154.18	154.19	332.74	-619.84	135.74	-253.35
1.. 2	0.00	462.29	84.51	127.34	127.34	262.11	-389.71	100.73	-159.29
1. 1	0.00	231 14	42.26	77.08	77.08	155.69	-187.59	63.50	-76.63
1 0	[illegible]	[illegible]	0.00	0.00	0.00	0.00	0.00	0.00	0.00

***BURMA DH.(SPAN 69.5+100.0+69.5 PIER 19.937)

SHEARING-FORCE

	S	SS	P +	P -	Q +	Q -	P1	Q1+	Q1-	Q2
1.. 0	469.36	83.13	37.32	-9.71	82.10	-34.01	-4.51	32.99	-13.99	0.00
1.. 1	298.26	51.26	32.08	-9.71	65.22	-35.28	-4.51	26.21	-14.50	0.00
1.. 2	147.91	19.27	26.99	-10.33	50.85	-39.07	-4.51	20.43	-16.03	0.00
1.. 3	1.94	-12.66	22.20	-15.12	38.84	-45.27	-4.51	15.62	-18.52	0.00
1.. 4	-157.67	-44.59	17.91	-19.51	29.15	-53.69	-4.51	11.71	-21.90	0.00
1.. 5	-312.36	-76.51	13.87	-23.45	21.44	-64.15	-4.51	8.61	-26.10	0.00
1.. 6	-473.46	-109.44	10.38	-26.94	15.54	-76.41	-4.51	6.24	-31.03	0.00
1.. 7	-651.27	-140.37	7.31	-30.01	11.24	-90.26	-4.51	4.52	-36.60	0.00
1.. 8	-861.19	-172.30	4.61	-32.71	9.34	-105.52	-4.51	3.35	-42.73	0.00
1.. 9	-1091.69	-204.23	2.19	-35.13	6.69	-122.03	-4.51	2.69	-49.35	0.00
1..10	-1348.17	-235.16	1.99	-37.32	6.15	-139.66	-4.51	2.47	-56.45	0.00
1..10	-1214.59	-229.70	3.73	-36.45	11.54	-139.14	-8.47	4.64	-57.14	0.00
1.. 9	-1028.53	-205.73	3.73	-35.49	11.71	-126.55	-8.47	4.71	-51.96	0.00
1.. 8	-856.00	-183.76	3.73	-34.41	12.23	-114.31	-8.47	4.92	-46.92	0.00
1.. 7	-701.11	-160.79	3.73	-33.19	13.16	-102.48	-8.47	5.30	-42.05	0.00
1.. 6	-567.72	-137.82	4.65	-31.90	14.54	-91.10	-8.47	5.87	-37.37	0.00
1.. 5	-442.71	-114.85	6.25	-30.20	16.45	-80.25	-8.47	6.66	-32.91	0.00
1.. 4	-347.48	-91.88	8.11	-28.34	18.96	-70.00	-8.47	7.69	-28.69	0.00
1.. 3	-260.75	-68.91	10.28	-26.17	22.19	-60.46	-8.47	9.02	-24.77	0.00
1.. 2	-175.79	-45.94	12.73	-23.72	26.21	-51.73	-8.47	10.67	-21.17	0.00
1.. 1	-92.01	-22.97	15.42	-21.03	31.14	-43.90	-8.47	12.70	-17.95	0.00
1.. 0	0.00	-0.00	18.22	-18.23	37.03	-37.03	-8.47	15.12	-15.12	0.00

Bending Moment Diagram

Dead Load of Girder

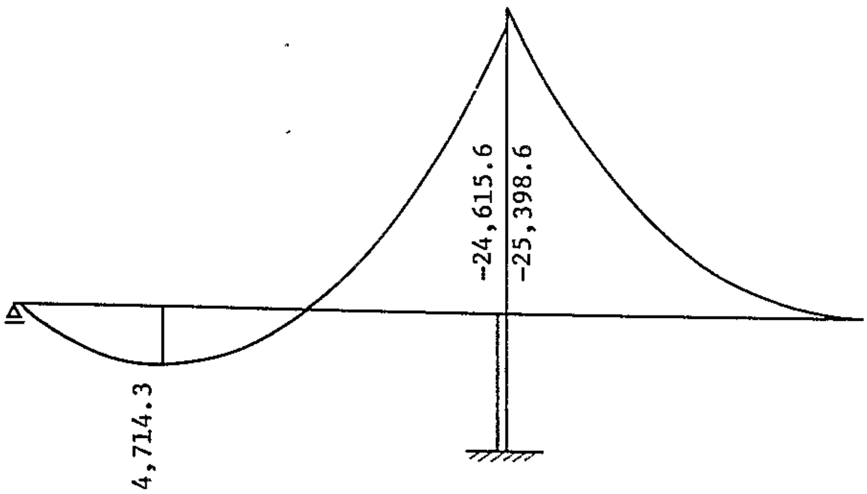


Fig. 5-16

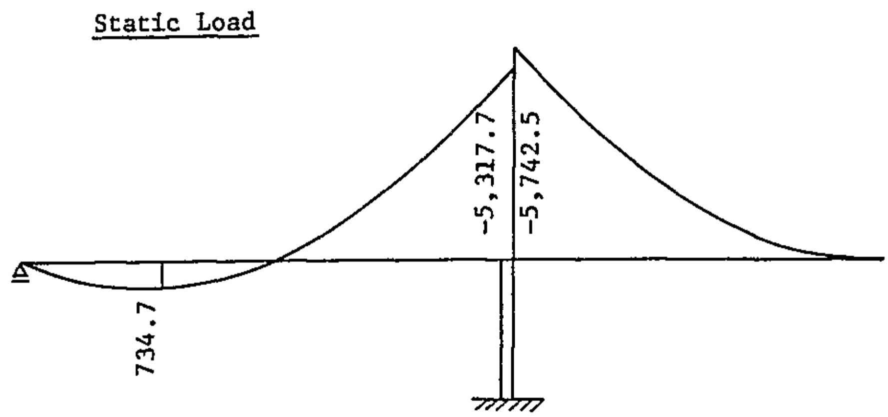


Fig. 5-17

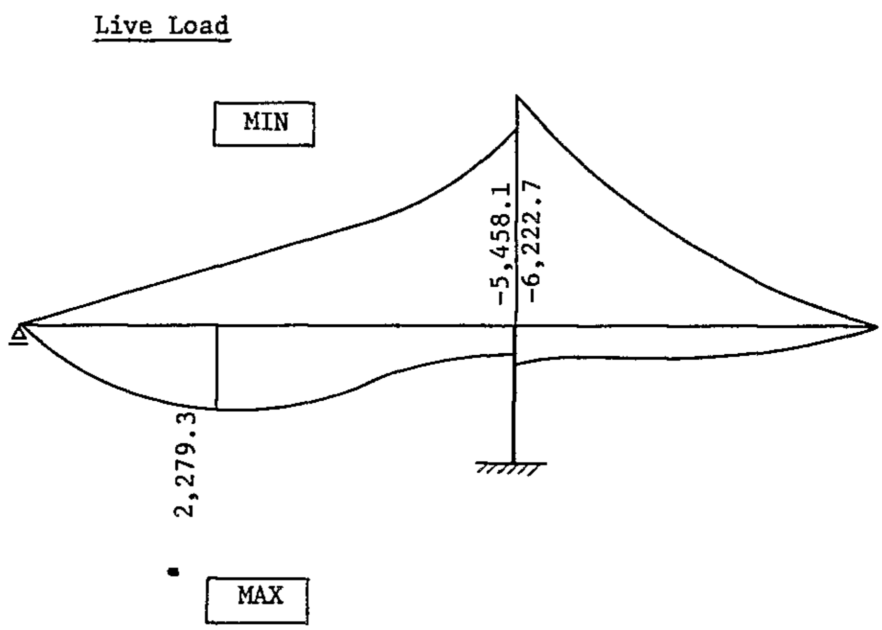


Fig. 5-18

(4) Initial Prestress

a) Determination of the average prestressing force

Decrement of tension of P.C. steel bars by the effect of friction.

$$Po = P \times \ell^{-(\mu\alpha + \lambda\ell)}$$

Where

Po: Tension of P.C. steel bar in the design section

P : Tension in the location of jack

μ : Frictional coefficient per radian of angular change

α : Angular change (in radian)

λ : Frictional coefficient per meter length of steel bar

ℓ : Length of steel bar

Steel bar used (SBPR 95/120 ø32)

$\mu = 0.30$, $\lambda = 0.003$, $P = 67.5t$

$\alpha = 30° = 0.5236$ radian -- bent up bar

According to the drawing, the length, number and tension of steel materials are as follows:

	ℓ (m)	N (piece)	Po (t)	
Straight bars	Below 10	6	8 m	65.9
	From 10 above and below 20	12	15	64.5
	20 - 30	9	25	62.6
	30 - 40	12	35	60.8
	40 - 50	7	45	59.0
	50 - 60	7	55	57.2
	60 - 70	8	65	55.5
	70 - 80	9	75	53.9
	Above 80	7	80	53.1
Bent up bars		16	50	49.7
Average value				57.7

GOHSEI ORYOKUDO INPUT DATA

EP" = 2,00E+07 EC" = 3,25E+06 ES" = 2,00E-04 AP" = 7,89E-04 PHI = 2,60E+00 XI = 3,00E-02

SECTION	PT	YP	N
1 L 0	57.70	2.181	0
1 L 1	57.70	2.358	66
1 L 2	57.70	2.573	70
1 L 3	57.70	2.725	70
1 L 4	57.70	2.553	68
1 L 5	57.70	1.757	76
1 L 6	57.70	0.805	88
1 L 7	57.70	0.303	104
1 L 8	57.70	0.275	128
1 L 9	57.70	0.321	164
1 L 10	57.70	0.371	198
1 R 10	57.70	0.371	198
1 R 9	57.70	0.296	158
1 R 8	57.70	0.262	140
1 R 7	57.70	0.219	114
1 R 6	57.70	0.208	98
1 R 5	57.70	0.174	74
1 R 4	57.70	0.168	62
1 R 3	57.70	0.163	38
1 R 2	57.70	0.163	26
1 R 1	57.70	0.164	10
1 R 0	57.70	0.000	0
2 L 0	57.70	0.000	0
2 L 1	57.70	0.164	10
2 L 2	57.70	0.163	26
2 L 3	57.70	0.163	38
2 L 4	57.70	0.168	62
2 L 5	57.70	0.174	74
2 L 6	57.70	0.208	98
2 L 7	57.70	0.219	114
2 L 8	57.70	0.262	140
2 L 9	57.70	0.296	158
2 L 10	57.70	0.371	198
2 R 10	57.70	0.371	198
2 R 9	57.70	0.321	164
2 R 8	57.70	0.275	128
2 R 7	57.70	0.303	104
2 R 6	57.70	0.805	98
2 R 5	57.70	1.757	76
2 R 4	57.70	2.553	68
2 R 3	57.70	2.725	70
2 R 2	57.70	2.573	70
2 R 1	57.70	2.358	66
2 R 0	57.70	2.181	0

b) Secondary reaction due to eccentricity of prestress

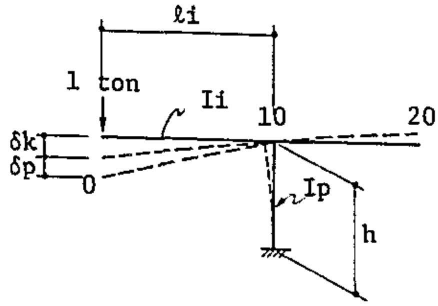


Fig. 5-19

Ek : Young's modulus of main girder

Ep : Young's modulus of pier

δK (deflection of main girder)

Section	Ii	$\overline{M}$	Mp	n	$n \cdot \overline{M}^2/I$	$n\overline{M}$ Mp/I
0	:	0	:	1	:	:
1	:	$1 \cdot \Delta x$	:	4	:	:
2	:	$2 \cdot \Delta x$	:	2	:	:
:	:	:	:	4	:	:
:	:	:	:	:	:	:
:	:	:	:	4	:	:
10	:	$10 \cdot \Delta x$	:	1	:	:

$\overline{M}$: Moment incurred by unit load

Mp: Eccentric moment of prestress

n : Simpson modulus

Deflection of main girder caused by unit load:

$$\delta k = \frac{\Delta x}{3} \; \Sigma \, n\overline{M}^2/I$$

Deflection of main girder caused by prestressing:

$$\delta kp = \frac{\Delta x}{3} \; \Sigma n\overline{M}Mp/I$$

Deflection of main girder due to displacement of pier under unit load:

$$\theta p = \frac{M10}{Ip} \cdot h \cdot \frac{Eu}{Ep}$$ - Angle of deflection of pier

$$\delta p = \theta p \cdot \ell i$$ - Deflection of main girder due to displacement of pier

Prestressing does not cause displacement of the pier.

Secondary reaction due to prestressing

Deflection due to prestressing

$$Xp_2 = \frac{\delta kp}{\delta_k + \delta_p}$$

Deflection incurred by unit load

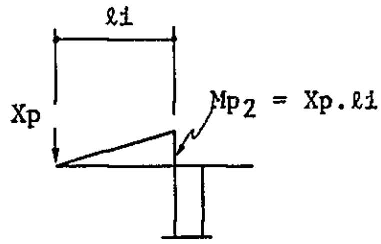
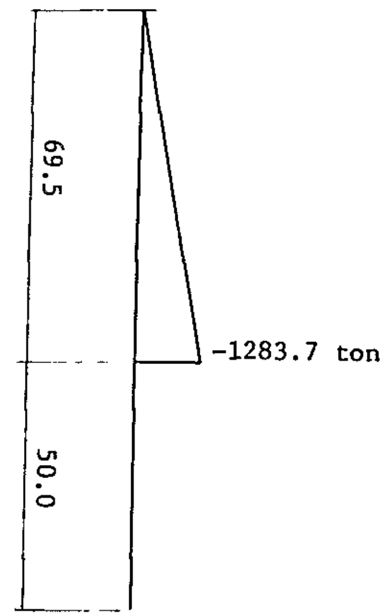


Fig. 5-20

DOHNYU PRE-STRESS NI YORU JIKURYOKU,HENSHIN MOMENT, 2-JI MOMENT

SECTION	PT	MPT1	MPT2
1 L 0	0.000	0.000	0.000
1 L 1	3808.200	-4620.271	-128.368
1 L 2	4039.000	-5973.118	-256.735
1 L 3	4039.000	-6339.499	-385.103
1 L 4	3923.600	-5112.141	-513.471
1 L 5	4385.200	-1668.807	-641.838
1 L 6	5077.600	3691.235	-770.206
1 L 7	6000.800	9047.890	-898.574
1 L 8	7385.600	14028.671	-1026.941
1 L 9	9462.800	20778.565	-1155.309
1 L 10	10847.600	26962.618	$\boxed{-1283.677}$
1 L XXP2 =	-18.470		
1 R 10	10847.600	26962.548	0.000
1 R 9	9116.600	20554.607	0.000
1 R 8	8078.000	16095.547	0.000
1 R 7	6577.800	11515.981	0.000
1 R 6	5654.600	8194.128	0.000
1 R 5	4269.800	4979.344	0.000
1 R 4	3577.400	3182.233	0.000
1 R 3	2192.600	1752.858	0.000
1 R 2	1500.200	1096.680	0.000
1 R 1	577.000	397.450	0.000
1 R 0	0.000	0.000	0.000
1 R XXP2 =	0.000		

Diagram of Secondary
Moment incurred by Prestressing



(5) Equivalent section (Section 10)

Increment of section by converting the section of PC steel bars into equivalent section of concrete.

$$A = Np.Ap(\frac{Ep}{Ec} - 1)$$
$$= 188 \times 0.000789 \times (\frac{20\ E6}{3.25\ E6} - 1)$$
$$= 0.7645\ m^2$$

	$A\ (m^2)$	y (m)	$Ay(m^3)$	$Ay^2(m^4)$	$Io(m^4)$
Total section	15.4110		44.02279	176.20010	22.93086
Section of steel material	0.7645	0.3710	0.28363	0.10523	
Converted equivalent area	16.1755		44.30642	176.30333	22.93086

$$Yo = \frac{44.30642}{16.1755} = 2.7391\ m$$

$$Yu = 5.800 - 2.7391 = 3.0609\ m$$

$$I = 176.30533 + 22.93086 - 16.1755 \times 2.7391^2$$
$$= 77.8768\ m^4$$

$$Zo = \frac{77.8768}{2.7391} = 28.4315\ m^3$$

$$Zu = \frac{77.8768}{3.0609} = 25.4425\ m^3$$

(6) Calculation of the effective prestress

Calculation of the effectiveness coefficient

$$\eta = 1 - (\frac{\sigma p\Psi}{\sigma pt} + r)$$

Where,

η : Effectiveness coefficient

σpt: Initial tensile stress of PC steel bar

$\sigma p\Psi$: Decrement of stress of steel bar due to creep and shrinkage

KANZAN DANMEN SEINO

SECTION	AK	IK	YOK	YUK	ZOK	ZUK
1 L 0	10.5910	11.4014	1.2595	-1.5405	9.0521	-7.4013
1 L 1	9.2559	10.3246	1.1799	-1.6234	8.7502	-6.3597
1 L 2	8.6029	10.1164	1.1431	-1.7271	8.8502	-5.8574
1 L 3	8.7655	11.4622	1.2064	-1.8144	9.5013	-6.3175
1 L 4	9.0133	13.3579	1.2901	-1.9607	10.3545	-6.8129
1 L 5	9.3799	16.0796	1.3890	-2.1646	11.5755	-7.4286
1 L 6	9.9315	20.5613	1.5055	-2.4151	13.6574	-8.5139
1 L 7	11.0152	29.4449	1.7529	-2.5884	16.7979	-11.3757
1 L 8	12.9396	42.9520	2.0991	-2.7057	20.4723	-15.8747
1 L 9	14.7796	59.2231	2.4177	-2.5769	24.4953	-20.5854
1 L 10	16.1754	77.8762	2.7391	-3.0609	28.4312	-25.4423
1 R 10	16.1755	77.8762	2.7391	-3.0609	28.4312	-25.4423
1 R 9	15.0017	60.1945	2.4541	-2.8297	24.5243	-21.2689
1 R 8	13.9144	45.5591	2.1724	-2.6078	20.9715	-17.4701
1 R 7	12.0013	33.1541	1.9021	-2.3997	17.4302	-13.8159
1 R 6	10.2026	23.0995	1.6005	-2.2598	14.4326	-10.2219
1 R 5	8.5195	15.3842	1.2990	-2.1676	11.8433	-7.0975
1 R 4	7.2713	10.0566	1.0267	-2.1035	9.7951	-4.7809
1 R 3	7.0113	8.0533	0.9448	-1.9149	8.5236	-4.2057
1 R 2	6.9436	6.7604	0.9527	-1.7788	7.6595	-3.8006
1 R 1	6.7050	6.0190	0.9496	-1.6920	7.0924	-3.5573
1 R 0	6.6410	5.7727	0.9371	-1.6609	6.8778	-3.4756

DOHNYU PRE-STRESS NI YORU ORYOKUDO

SECTION	CP1O	CP1U	CP2O	CP2U	CPO	CPU
1 L 0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
1 L 1	-108.3717	1194.5523	-14.7522	21.4177	-123.1539	1215.9700
1 L 2	-201.3300	1600.5514	-29.5238	47.9243	-230.8539	1648.4761
1 L 3	-203.1650	1572.6531	-41.2821	66.6129	-244.4251	1639.2660
1 L 4	-46.1975	1241.7490	-49.7474	79.6158	-95.9449	1321.3647
1 L 5	340.2512	710.0471	-55.0909	87.1360	285.1603	797.1830
1 L 6	813.4549	103.3279	-57.8990	90.2748	755.5559	193.6025
1 L 7	1140.9274	-236.3225	-57.0511	79.7278	1083.8754	-156.5946
1 L 8	1336.0121	-301.6749	-54.2565	65.6172	1281.7456	-236.0577
1 L 9	1601.8338	-357.3861	-51.7520	57.1524	1550.0459	-300.2337
1 L 10	1753.5590	-377.6997	-49.9743	51.4934	1703.5849	-326.1953
1 R 10	1753.5513	-377.6900	0.0000	0.0000	1753.5513	-377.6909
1 R 9	1553.7295	-349.6941	0.0000	0.0000	1553.7295	-349.6841
1 R 8	1446.1594	-326.9883	0.0000	0.0000	1446.1594	-326.9883
1 R 7	1283.6913	-274.7449	0.0000	0.0000	1283.6913	-274.7448
1 R 6	1185.7905	-232.9721	0.0000	0.0000	1185.7905	-232.9721
1 R 5	964.7857	-196.7676	0.0000	0.0000	964.7857	-196.7676
1 R 4	850.9353	-159.0142	0.0000	0.0000	850.6353	-159.0142
1 R 3	531.7942	-99.1978	0.0000	0.0000	531.7942	-98.1978
1 R 2	369.3830	-96.4520	0.0000	0.0000	369.3830	-66.4520
1 R 1	143.0612	-75.2405	0.0000	0.0000	143.0612	-25.2405
1 R 0	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

YUKOH KEISU AND ETA

SECTION	CCPT	CSPT	CCP	CSP	CPPHI	ETA
1 R 0	0.0000	73130.5450	0.0000	0.0000	4000.0000	0.9153
1 R 1	1003.2191	73130.5450	630.6252	-372.5939	11799.0401	0.8087
1 R 2	1453.9819	73130.5450	711.5273	-742.3546	12006.0771	0.8059
1 R 3	1454.9493	73130.5450	685.0537	-769.7655	11674.1845	0.8104
1 R 4	1017.1642	73130.5450	558.6309	-458.4833	10810.6619	0.8222
1 R 5	539.3233	73130.5450	479.2763	-59.0469	10567.4233	0.8255
1 R 6	640.1715	73130.5450	609.6109	-31.5605	12223.3051	0.8029
1 R 7	997.2092	73130.5450	699.9221	-297.3761	12739.7395	0.7958
1 R 8	1194.9559	73130.5450	645.0456	-549.8104	11031.0012	0.8110
1 R 9	1437.9713	73130.5450	661.2144	-776.6569	11405.4294	0.8140
1 R 10	1573.7498	73130.5450	567.5914	-1006.1574	10027.2842	0.8329
1 R 10	1617.2253	73130.5450	567.9053	-1049.3200	9966.8490	0.8337
1 R 9	1447.0990	73130.5450	483.0178	-964.0813	9162.1871	0.8467
1 R 8	1348.9751	73130.5450	486.3669	-862.6082	9342.6550	0.8422
1 R 7	1204.3538	73130.5450	427.9697	-776.3839	8796.9954	0.8497
1 R 6	1109.3451	73130.5450	445.4007	-663.9445	9159.7631	0.8447
1 R 5	906.9846	73130.5450	358.0223	-548.9624	8275.6504	0.8568
1 R 4	796.6377	73130.5450	384.9543	-411.6834	8802.1303	0.8496
1 R 3	495.8856	73130.5450	231.3712	-264.4944	7027.7710	0.8739
1 R 2	342.2216	73130.5450	208.7313	-133.4904	6883.7591	0.8759
1 R 1	132.1972	73130.5450	92.5307	-39.6664	5343.7675	0.8969
1 R 0	0.0000	73130.5450	0.0000	0.0000	4000.0000	0.9153

$$\sigma p\psi = \frac{n.\psi.\sigma cp + Ep.\ s}{1 + n\,\dfrac{\sigma cpt}{\sigma pt}\,(1 + \dfrac{\psi}{2})}$$

n : ratio of young's modulus ($Ep/Ec = 20\times10^6/3.25\times10^6$)

ψ : Creep modulus (2.6)

εs : Rate of drying shrinkage (2.0×10^{-4})

σcp: Concrete stress at the centroid of PC steel bar incurred by deal load

σcpt: Initial concrete stress at the centroid of PC steel bar incurred by prestressing.

γ : Relaxation (0.03)

Section 10 (Side Span)

1. Stress at the centroid of the PC steel bar

a) Prestress introduced

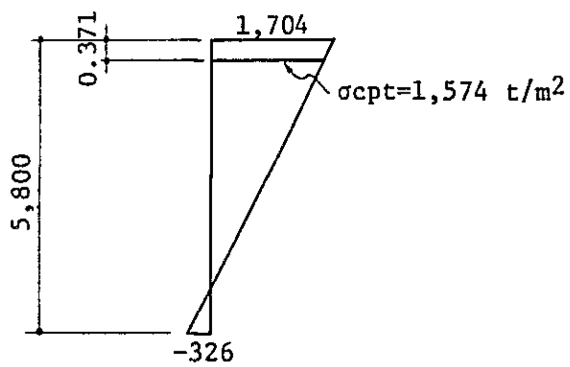


Fig. 5-21

b) Due to dead load

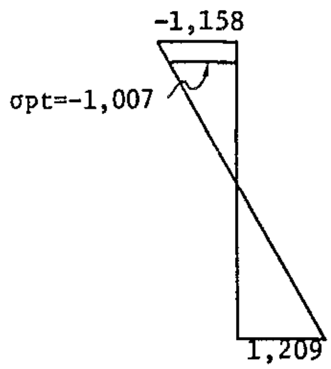


Fig. 5-22

c) Stress due to dead load

$$\sigma cp = \sigma cpt + \ cpt$$

$$= 1574 - 1007 = 567 \ t/m^2$$

2. Tensile Stress of PC Steel Bar

Tensile force of PC steel bar Pt = 57.7 t/piece

Section of PC steel bar Ap = 7.89×10^{-4} m^2

$$\sigma pt = \frac{Pt}{Ap} = \frac{57.7}{1.89 \times 10^{-4}} = 73131 \ t/m^2$$

3. Decrement of Stress of Steel Bar due to Creep and Drying Shrinkage

$$\sigma py = \frac{6.154 \times 2.6 \times 576 + 20 \times 10^6 \times 2.0 \times 10^{-4}}{1 + 6.154 \times \frac{1574}{73131} \times (1 + \frac{2.6}{2})}$$

$$= \frac{9072 + 4000}{1 \ + 0.305}$$

$$= 10017 \ t/m^2$$

4. Effectiveness Coefficient

$$\eta = 1 - (\frac{10017}{73131} + 0.03) = 0.8330$$

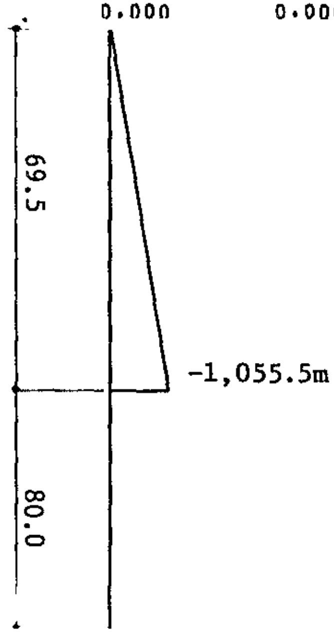
YUKOH PRE-STRESS NI YORU STRESS

SECTION			PE	MPE1	MPE2	CPEO1	CPEU1	CPEO2	CPEU2	CPEO	CPEU
1	L	0	0.000	0.000	0.000	0.000	0,000	0.000	0,000	0,000	0,000
1	L	1	3079.531	-3736.218	-105.546	-87.636	965,984	-12,154	17,610	-99,790	983,594
1	L	2	3254.734	-4813.299	-211.091	-162.237	1289,768	-24,275	39,404	-186,512	1329,172
1	L	3	3273.065	-5137.309	-316.637	-164.639	1274,423	-33,926	54,770	-198,565	1329,193
1	L	4	3225.878	-4203.065	-422.183	-37,982	1020,933	-40,903	65,461	-78,885	1086,394
1	L	5	3619.979	-1377.599	-527.729	280.877	586,143	-45,297	71,644	235,581	657,788
1	L	6	4076.583	2963.531	-633.274	653.087	82.957	-47,605	74,225	605,482	157,183
1	L	7	4775.404	7200.261	-738.820	907.944	-188,064	-46,908	65,553	861,036	-122,511
1	L	8	5989.394	11376.630	-844.366	1083.447	-244,645	-44,619	53,951	1038,828	-190,694
1	L	9	7703.099	16914.586	-949.912	1303.955	-290,927	-42,576	46,992	1261,379	-243,935
1	L	10	9034.805	22455.764	$\boxed{-1055.457}$	1460.513	-314.571	-41,090	42,339	1419,424	-272,233

1 L XPE2 = -15.186

1	R	10	9043.770	22478.989	0.000	1461.956	-314,885	0,000	0,000	1461,956	-314,885
1	R	9	7700.925	17362.777	0.000	1312.458	-295,383	0,000	0,000	1312,458	-295,383
1	R	8	6803.670	13556.424	0.000	1218.023	-275,405	0,000	0,000	1218,023	-275,405
1	R	7	5589.211	9785.225	0.000	1090.763	-233,453	0,000	0,000	1090,763	-233,453
1	R	6	4776.711	6921.972	0.000	1001.694	-196,803	0,000	0,000	1001,694	-196,803
1	R	5	3658.524	4266.487	0.000	826.664	-160,029	0,000	0,000	826,664	-160,029
1	R	4	3039.495	2703.747	0.000	722.903	-135,105	0,000	0,000	722,903	-135,105
1	R	3	1916.115	1531.824	0.000	464.736	-85,806	0,000	0,000	464,736	-85,806
1	R	2	1313.981	960.549	0.000	323.094	-58,203	0,000	0,000	323,094	-58,203
1	R	1	517.529	356.484	0.000	128.316	-22,639	0,000	0,000	128,316	-22,639
1	R	0	0.000	0.000	0.000	0.000	0,000	0,000	0,000	0,000	0,000

1 R XPE2 = 0,000



(7) Composite stress

SUM OF STRESS

SECTION			CSUMD 10	CSUMU 10	CSUMD 11	CSUMU 11	CSUMD 12	CSUMU 12
1	L	0	0.000	0.000	0.000	0.000	0,000	0,000
1	L	1	361.850	326.420	209.221	536.420	235,055	500,875
1	L	2	552.364	159.395	278.546	573.120	329,631	495,934
1	L	3	591.732	85.902	247.597	603.468	318,973	496,122
1	L	4	584.269	46.986	209.189	617.042	296,515	484,322
1	L	5	645.449	14.310	276.133	589.840	373,768	437,689
1	L	6	689.254	21.563	363.119	544.734	462,429	385,425
1	L	7	601.317	239.367	337.700	628.637	431,901	489,535
1	L	8	464.645	505.584	245.919	790.957	340,976	668,370
1	L	9	402.620	714.462	198.310	957.576	306,357	829,008
1	L	10	287.259	909.345	69.381	1151.851	199,779	1006,135
1	R	10	289.721	907.528	35.854	1191.219	336,020	855,790
1	R	9	257.919	947.043	8.418	1134.733	305,653	792,003
1	R	8	281.524	784.559	35.651	1079.714	329,291	727,217
1	R	7	262.471	756.185	14.924	1068.493	309,371	697,015
1	R	6	296.301	748.353	46.616	1100.890	337,814	689,739
1	R	5	260.950	742.104	9.172	1162.237	292,805	688,949
1	R	4	308.707	578.295	62.967	1181.781	327,328	640,145
1	R	3	223.813	359.171	6.826	827.933	227,436	380,826
1	R	2	227.905	129.531	60.913	465.036	220,115	144,229
1	R	1	122.199	11.046	27.864	177.035	111,786	9,716
1	R	0	0.000	0.000	0.000	0.000	0,000	0,000

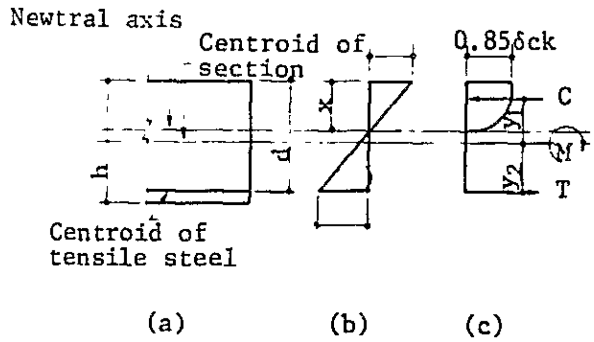
SUM OF STRESS

SECTION			CSUMD 13	CSUMU 13	CSUMD 14	CSUMU 14	CSUMD 15	CSUMU 15
1	L	0	0.000	0.000	0.000	0.000	0,000	0,000
1	L	1	275.071	445.918	367.188	319.076	203,884	543,763
1	L	2	408.757	376.378	562.918	143.449	267,992	589,067
1	L	3	429.529	329.949	606.479	63.724	232,851	625,647
1	L	4	431.778	278.745	602.311	19.565	191,147	644,463
1	L	5	524.998	202.017	665.621	-17.125	255,962	621,275
1	L	6	616.255	138.665	709.772	-11.351	342,601	577,648
1	L	7	577.813	274.075	620.779	210.628	318,237	657,376
1	L	8	477.803	491.916	492.896	485.348	227,669	814,493
1	L	9	435.006	675.925	419.750	694.043	181,150	977,994
1	L	10	322.933	969.513	303.716	889.988	52,954	1170,208
1	R	10	173.421	1037.491	304.583	890.920	20,991	1207,828
1	R	9	136.002	987.621	273.427	829.162	-7,090	1152,614
1	R	8	152.944	935.911	297.643	765.209	19,532	1099,064
1	R	7	123.715	931.240	279.441	734.775	-2,046	1089,902
1	R	6	145.630	961.090	313.868	723.550	29,049	1125,693
1	R	5	97.636	1014.520	278.790	712.336	-8,668	1192,006
1	R	4	138.544	1026.935	325.963	642.941	45,711	1217,136
1	R	3	64.728	710.584	238.685	358.029	-8,046	858,075
1	R	2	99.390	387.502	238.940	106.295	49,878	487,273
1	R	1	46.605	139.669	128.157	22.924	21,906	188,914
1	R	0	0.000	0.000	0.000	0.000	0,000	0,000

(8) Study of the Bending Failure

The method of calculating of failure resistent bending moment is as follows:

a) By assuming the location of the neutral axis, connecting and extending the line between this and the ultimate strain of the compression edge of the section, the amount of strain at the centroid of the pc steel bars can be obtained. In the case of reinforcing bar, this amount can be taken as its deflection εo. However, in the case of pc steel bar, this amount should be added with the strain caused by the effective prestressing force to obtain the strain ε.



(a) Dimension of member

(b) Distribution of deflection

(c) Distribution of stress

Distribution of deflection and stress for the calculation of failure resistant bending moment.

Fig. 5-23

b) Obtain the stress corresponding to the deflection of pc steel bar ε from the stress - Strain cirve, and then calculate the resultant of the tensile force of the steel bar by using formula.

$$T = A \cdot \sigma s$$

where,

T : Resultant of the tensile force of the steel bar (kg)

A : Total section of tensile steel bar (cm^2)

σs : Stress corresponding to the total strain of the tensile steel bar ε.

c) Further, the resultant force C of the compressive stress of of concrete can be obtained by using the following formula

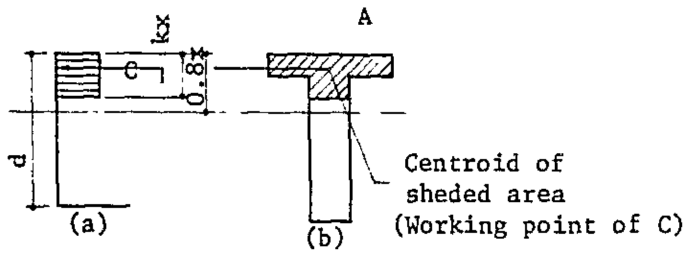
$$C = 0.85 \cdot \sigma ck \cdot As$$

Where,

C: Resultant force of compressive stress of concrete (kg)

σck: Design standard strength of concrete (kg/cm^2)

A : Area of section over which compressive stress distributes (area in following figure (b)) (cm^2)



Area (working point of C)

(a) Distribution of stress

(b) Cross section of member

Working point of compressive stress of concrete.

Fig. 5-24

d) Where the equality T=C is not satisfied, reassume the location of the neutral axis and repeat the process 1 - 3 until the equilibrium is satisfied.

e) When T = S is satisfied, calculate the failure resistant bending movement by using following

$M = C. + T.$

where,

Y_1, Y_2: Distance between centroid of the section and point C and T. (cm)

f) Calculation of the Failure Resistant Moment and the Safety Factor

Side span

SEC	C=T (T)	YX (M)	YT (M)	YC (M)	MP (TM)	MU (TM)	KD	SF
0	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
1	4999.1	0.181	2.358	0.072	11424.2	6724.6	1.7	1.70
	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
2	5302.1	0.192	2.573	0.077	13233.0	10773.8	1.7	1.23
	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
3	5302.1	0.192	2.725	0.077	14038.6	12299.0	1.7	1.14
	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
4	4962.9	0.180	2.643	0.072	12762.0	11229.2	1.7	1.14
	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
5	4924.9	0.178	2.026	0.071	9626.8	8386.7	1.3	1.15
	4667.8	-0.302	-2.187	-0.121	-9643.3	-3049.5	1.0	3.16
6	5061.2	0.183	1.007	0.073	4725.1	3576.7	1.0	1.32
	6066.3	-0.706	-3.411	-0.179	-19607.0	-7831.2	1.3	2.50
7	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	7877.4	-0.917	-4.038	-0.234	-29969.8	-15237.0	1.7	1.97
8	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	9695.2	-0.758	-4.529	-0.254	-41446.3	-27121.7	1.7	1.53
9	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	12422.0	-1.009	-4.975	-0.328	-57724.3	-42501.0	1.7	1.36
10	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	14239.9	-1.028	-5.429	-0.370	-72031.1	-617[illegible]3.4	1.7	1.17

Resistant Moment

Central span

SEC	C=T(T)	YX(M)	YT(M)	YC(M)	MR(TM)	MU(TM)	KD	SF
0	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
1	757.4	0.027	0.600	0.011	446.2	416.5	1.0	1.07
	757.4	-0.049	-1.941	-0.020	-1455.4	-1360.0	1.3	1.07
2	1969.3	0.071	0.163	0.029	264.7	75.1	1.0	3.53
	1969.3	-0.127	-2.499	-0.051	-4821.1	-3491.1	1.3	1.38
3	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	2879.3	-0.186	-2.697	-0.074	-7548.6	-6522.7	1.7	1.16
4	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	4696.1	-0.690	-2.962	-0.143	-13237.1	-10674.1	1.7	1.24
5	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	5605.1	-0.365	-3.293	-0.145	-17645.2	-15887.7	1.7	1.11
6	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	7422.9	-0.480	-3.652	-0.192	-25683.7	-22407.7	1.7	1.15
7	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	8634.8	-0.558	-4.083	-0.223	-33328.1	-30313.2	1.7	1.10
8	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	10404.2	-0.686	-4.518	-0.274	-45001.5	-39803.1	1.7	1.13
9	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	11967.6	-0.774	-4.988	-0.309	-55990.7	-51064.3	1.7	1.10
10	0.0	0.000	0.000	0.000	0.0	0.0	0.0	0.00
	14239.9	-1.028	-5.429	-0.370	-72037.8	-64237.9	1.7	1.12

(9) Study of Shearing Force

a) Calculation of average shearing stress and principal tensile stress

Design Process:

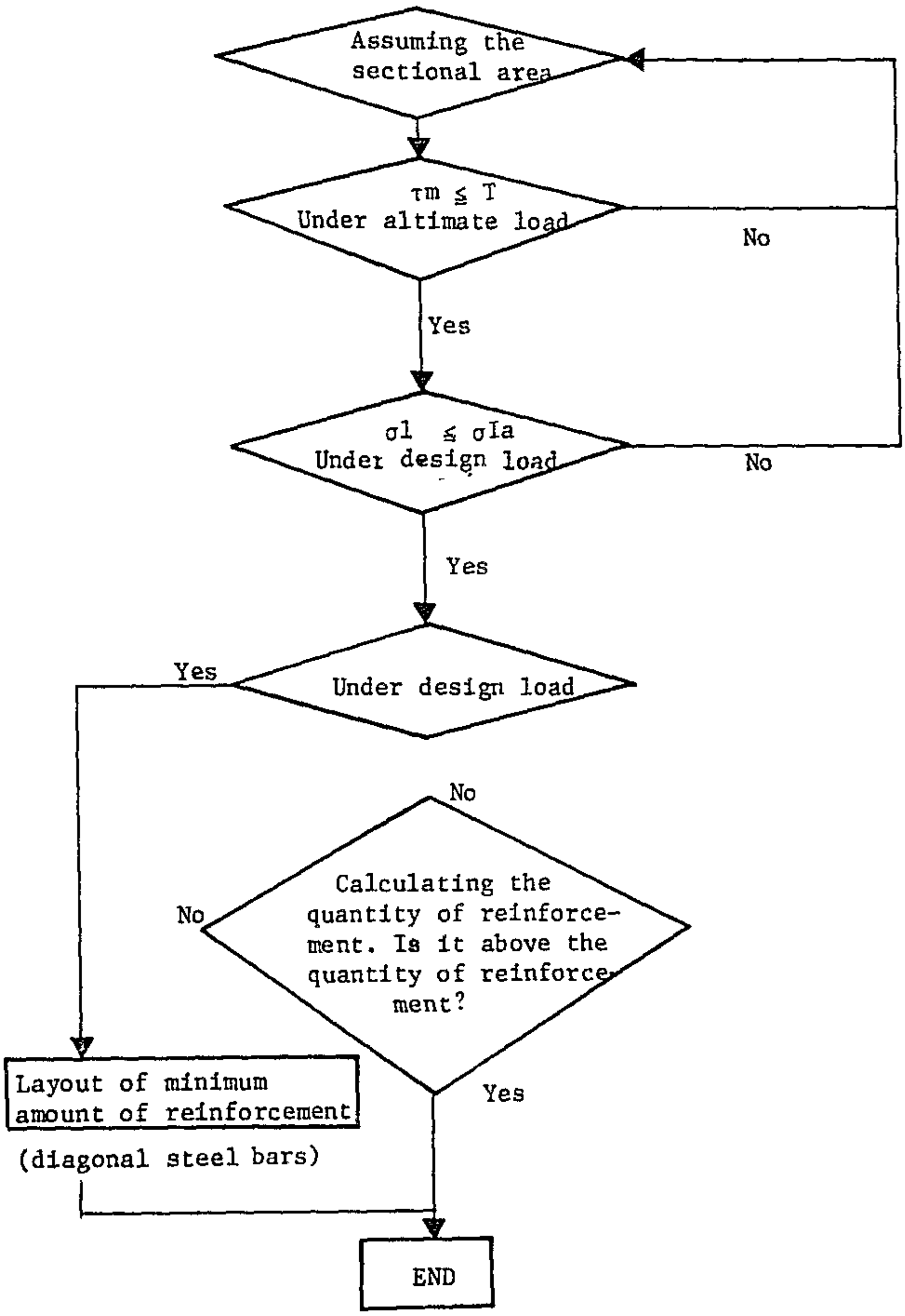


Fig. 5-25

b) Design of the diagonal pc steel bar

On the design of diagonal or verticle pc steel bars:

In cases where the required quantity of diagonal tensile reinforcing bar calculated based on following formula is so large that laying out of such bars becomes difficult, substitution of such reinforcement can be done by placing diagonal steel bars which form angles of 45° or more with the axis of the member, or verticle bars. In both cases at smaller interval than 1/2 of the effective height of the member, joining to the web. (See following figure) The required quantity of steel bar can be calculated from the formula

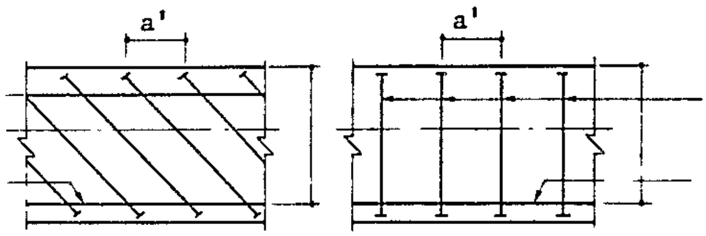


Fig. 5-26 Layout of diagonal or verticle steel bars

$$Ap = \frac{1.15\ Shp' \cdot a'}{d\ -\sigma s\ (\sin\theta' + \cos\theta')}$$

$$\sigma s = \sigma pe + \sigma sy \leq \sigma py$$

$$Shp' = Sh - Sp - Sc - Ss$$

Where,

Shp' : Shearing force born by diagonal steel bar placed at interval a' and indication angle ' or by verticale steel bars. (kg)

σpe : Effective tensile stress of diagonal or verticle steel bars (kg/cm^2)

σsy : Yielding point of the diagonal tensile reinforcement (kg/cm^2)

σpy : Yielding point of the diagonal or verticle PC steel bar (kg/cm^2)

Sp : Component of effective prestress force, in the direction of shearing force, of the P.C. steel bar placed in the axial direction. (Excluding component force of diagonal steel bar.)

Sh : Shearing force under ultimate load, with the effect of change in depth of the member taken into account. (kg)

Ss : Shearing force born by diagonal tensile reinforcement (kg)

a' : Spacing of diagonal or verticles steel bars in the axial direction of the member.

θ' : Angle between the diagonal or verticle steel bar and axis of the member.

Required Pitch of Diagonal Steel Bars under Ultimate and Design Load

◆◆◆ NEED PITCH ◆◆◆ (Side span)

SECT	PITCH		SENDAN NO JOOTAI	REMARK	AL(P) ◆∂-4	AL(S) ◆∂-4	REM
A1 0*	0.5766	-	HAKAIJI	[F=1 C=5 C,S & P]	0.00	0.00	S
1	1.2037	-	HAKAIJI	[F=1 C=5 C,S & P]	0.00	0.00	S
2	1.4350	-	HAKAIJI	[F=1 C=1 C,S & P]	0.00	0.00	S
3	1.5105	-	HAKAIJI	[F=1 C=2 C,S & P]	0.00	0.00	S
4	1.4173	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S
5	1.1187	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S
6	0.8633	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S
7	0.6018	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S
8	0.5346	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S
9	0.4933	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S
P1 10*	0.4694	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	S

KAJUU CASE NUMBER

HAKAIJI	JOOJI
1.3D+2.5L (1:MAX 2:MIN)	1.0D+1.0L (1:MAX 2:MIN)
1.0D+2.5L (3:MAX 4:MIN)	
1.7D+1.7L (5:MAX 6:MIN)	

◆◆◆ NEED PITCH ◆◆◆ (Central span)

SECT	PITCH		SENDAN NO JOOTAI	REMARK	AL(P) ◆∂-4	AL(S) ◆∂-4	REM
℄ 0	1.2500	-	HAKAIJI	[F=1 C=2 C,S & P]	0.00	0.00	
1	1.2705	-	JOOJI	[F=1 C=2 UPPER]	0.00	0.00	
2	0.8770	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	
3	0.5050	-	JOOJI	[F=1 C=2 UPPER]	0.00	0.00	
4	0.5141	-	JOOJI	[F=1 C=2 UPPER]	0.00	0.00	
5	0.4871	-	JOOJI	[F=1 C=2 UPPER]	0.00	0.00	
6	0.6932	-	JOOJI	[F=1 C=2 UPPER]	0.00	0.00	
7	0.7504	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	
8	0.8003	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	
9	0.7518	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	
P1 10*	0.7445	-	HAKAIJI	[F=1 C=6 C,S & P]	0.00	0.00	

KAJUU CASE NUMBER

HAKAIJI	JOOJI
1.3D+2.5L (1:MAX 2:MIN)	1.0D+1.0L (1:MAX 2:MIN)
1.0D+2.5L (3:MAX 4:MIN)	
1.7D+1.7L (5:MAX 6:MIN)	

5. Structural Design of Cross Section

Study of the direction perpendicular to the bridge axis will be made with overhanging slabs taken as cantilevered beems and the box section taken as the rigid frame. In the structural analisis of the rigid frame, the moment distribution method by G. Kani will be applied.

Bending stress will be studied with the upper slab taken as a P.C. member and the lower slab as a R.C. member.

For the section of the main girder which has a changing girder depth, comparative study of several alternative cases will be made.

Location of Section under Consideration

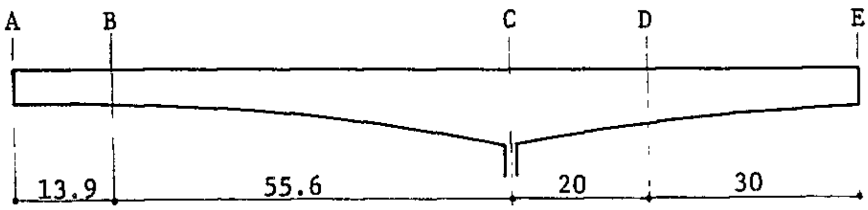


Fig. 5-27

a) Shape of each Section.

	Depth of girder H	Thickness of upper slab t1	Thickness of lower slab t2	Width of web W
A	2,800	0.300	0.500	0.800
B	2,870	"	0.250	0.550
C	5,800	"	0.700	0.700
D	3,860	"	0.400	0.485
E	2,500	"	0.250	0.300

b) Additional rate for bending moment due to live load

For bridges carrying over 1,000 heavy vehicle in one-way daily, the design bending moment of bridge slab corresponding to T load (inclusive of impact load) is to be taken at the value of the design bending moment multiplied by the increment factor in the following table.

Increment Factor for the Design Bending Moment
Correspondent to T load (inclusive of impact load)

Span of slab	Increment factor
$4.0 < 1 \leq 6.0$	$1.2 - (1 - 4)/30$

c) Spans for simply supported slab and continuous slab bearing T-load and dead load are to be taken as in the following figure.

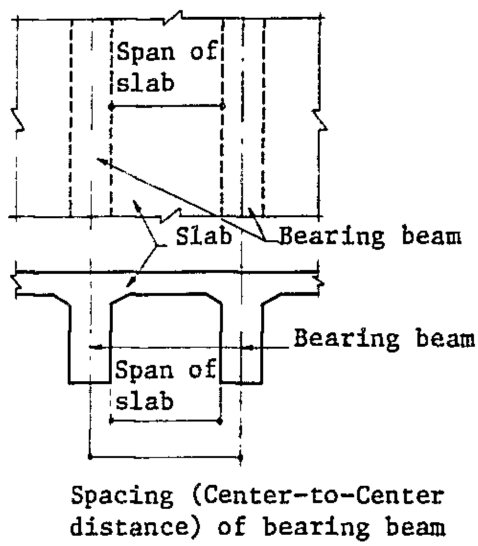


Fig. 5-28

The span of a overhanging slab bearing T-load and dead load is to be as given in the following figure.

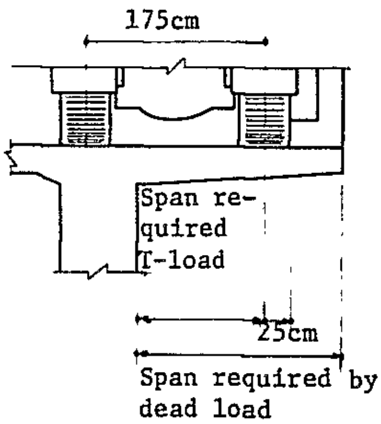


Fig. 5-29

d) Calculation of The Bending Moment

The deisgn bending moment per unit width (1m) of slab in responce to T-load (inclusive of impact load) is to be calculated based on formula given in the following table.

Design bending moment per unit width (1m) of slab in response to T-load (inclusive of impact load).

(kg.m/m)

Calculation of the Bending Moment

The design bending moment per unit width (1 m) of slab in response to T load (inclusive of impact load) is to be calculated based on criteria given in the following table.

Table-2 Design bending moment per unit width (1 m) of slab in response to T load (inclusive of impact load)

(kg.m/m)

Type of slab	Type of Bending moment	Structure	Direction of slab span / Direction of bending moment / Applicable area	Perpendicular to vehicle traffic: Direction of span	Perpendicular to vehicle traffic: Perpendicular direction to span	Parallel with vehicle traffic direction: Direction of span	Parallel with vehicle traffic direction: Perpendicular direction to span
Simple plate	Bending moment of span	RC	$0 \leq \ell \leq 4$	$+(0.12\ell + 0.07)P$	$+(0.10\ell + 0.04)P$	$+(0.22\ell + 0.08)P$	$+(0.06\ell + 0.06)P$
		PC	$0 \leq \ell \leq 6$				
Continuous plate	Bending moment of span	RC	$0 \leq \ell \leq 4$	+(Plus 80% of that of simple plate)	+(Plus 80% of that of simple plate)	+(Plus 80% of that of simple plate)	+(Plus 80% of that of simple plate)
		PC	$0 \leq \ell \leq 6$				
	Bending moment of sup port	RC	$0 \leq \ell \leq 4$	$-(0.15\ell + 0.125)P$	-	-(Minus 80% of that of simple plate	-
		PC	$0 \leq \ell \leq 6$				
Overhanging slab	Bending moment of sup-port	RC	$0 \leq \ell \leq 1.5$	$-P.\ell$	-	$-(0.7\ell + 0.22)P$	-
		PC	$0 \leq \ell \leq 1.5$	$1.30\ell + 0.25$			
			$1.5 \leq \ell \leq 3.0$	$-(0.6\ell - 0.22)P$			
	Bending moment at end of slab	RC	$0 \leq \ell \leq 1.5$	-	$+(0.15\ell + 0.13)P$	-	$+(0.16\ell + 0.07)P$
		PC	$0 \leq \ell \leq 3.0$				

Where RC : Reinforced concrete slab, PC : Prestressed concrete slab,
ℓ : Span of slab required by T load as stipulated in 5.3. (m)
P : The single rear wheel vehicle load as given in 1.8.4 of General Section.
For first class bridge : p = 8,000 kg
For second class bridge: p = 5,600 kg

Where,

RC : Reinforced concrete slab

PC : Prestressed concrete slab

l : Span of slab (m)

P : The single rear wheel vahicle load.

For first class bridge : P = 8000 kg.

For second class bridge : P = 5000 kg.

e) Checking of the stress of the lower flange and the web.

1) The stresses in the lower frange and the web are to be checked, as a rigid frame of which the upper and lower franges and the web constitute the structural member.

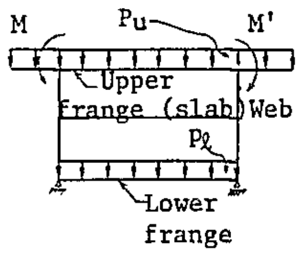


Fig. 5-30

Where,

Pu : Dead load of slab (kg/m)

P_ℓ : Dead load of lower frange (kg/m)

M, M' : Bending moment at the support in response to the live load calculated according to requirements provided in the above table.

Fig. 8-2-1 : Load condutions for checking the stress in the lower and the upper frange.

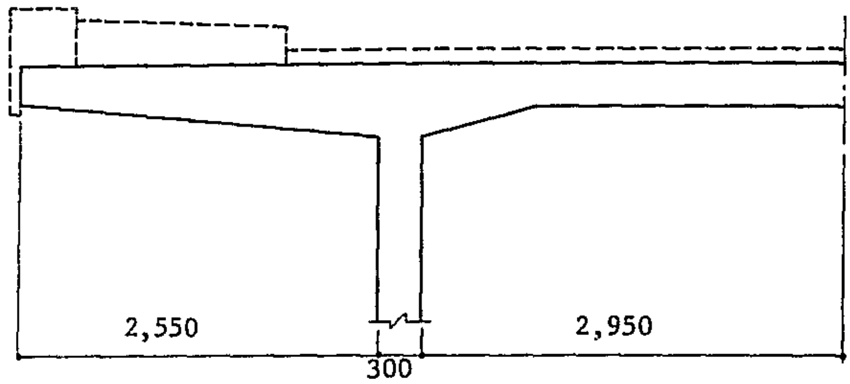
2) In checking the stress stated in item 1) above, the internal force can be calculated based on the load condition illustrated in Fig. 5.30.

The internal force occurred outside of the plane, in both cases of the lower frange and the web, can be obtained by calculation, taking the girder section as a box rigid frame, as illustrated in Fig. 5-30, in which each web is supported at the lower end to prevent it from settling.

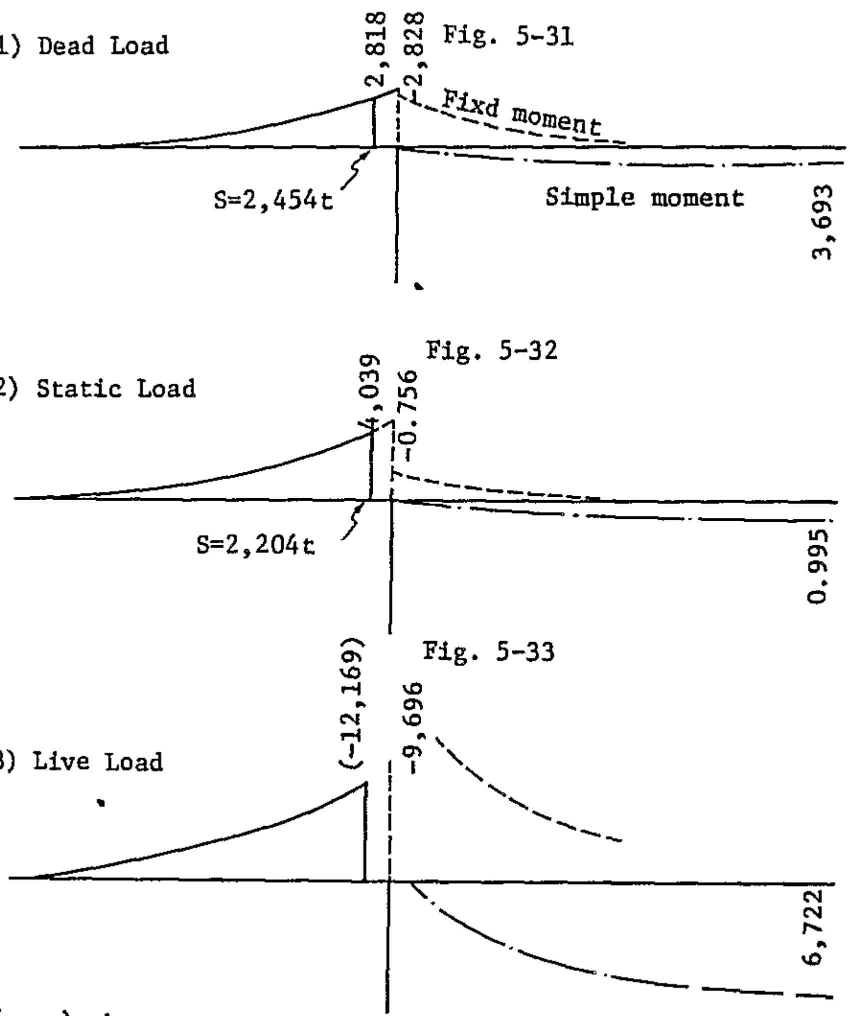
In this case, it is not necessary to consider simultaneously the effect bening moment of both the overhanging slab and the support of inner span incurred by the live load, since the load conditions have been separately chosen such that the bending moment obtained by calculation is max. respectively. Also, in the case where the upper frange is subject to prestressing the effect of prestress must be taken into consideration.

f) Design of slab

E-E Section



Bending moment of cantilevered slab



Figures ( ) show the value under collision load

Fig. 5-34

Bending moment of box frame

1 Dead Load

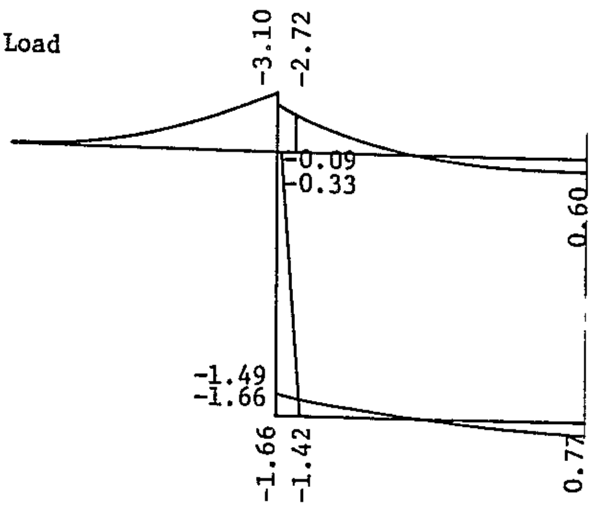


Fig. 5-35

2 Static Load

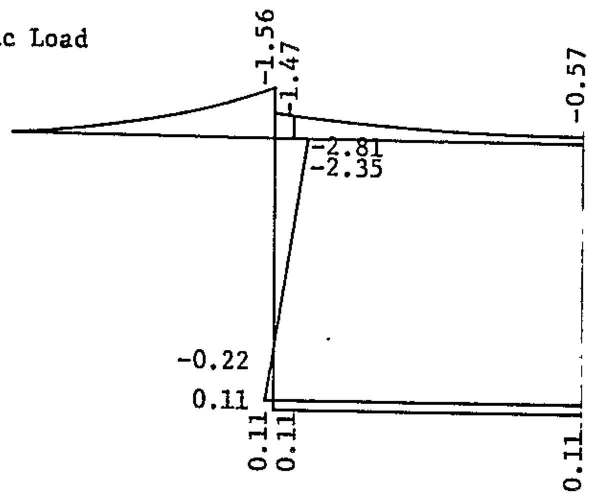


Fig. 5-36

3 Live Load

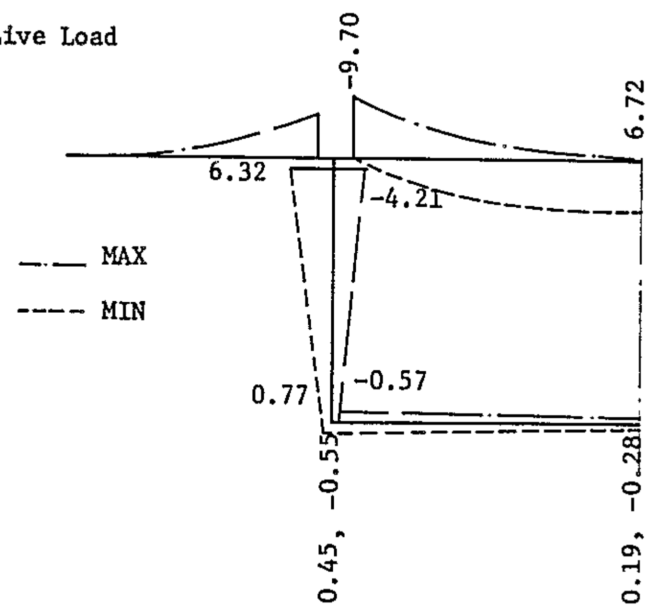


Fig. 5-37

g) Prestress

Location of P.C. Steel Bar

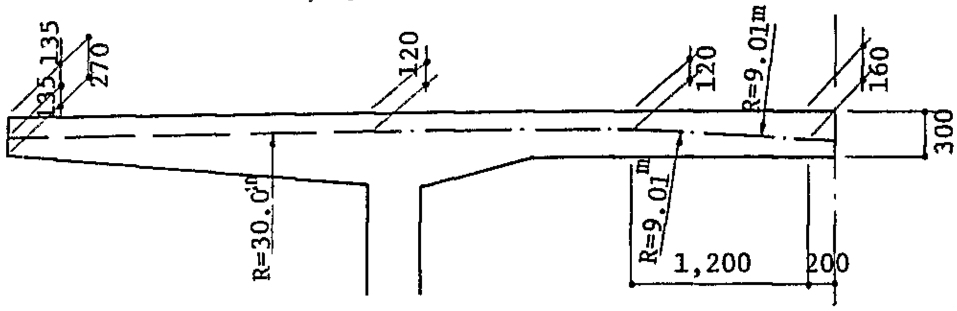


Fig. 5-38

P.C. steel bar	SBPR 95/120 ϕ32	
Tension introduced:		Pt = 56.7 ton per piece
Effective tension:		Pe = 48.2 ton per piece (effective coefficient η = 0.85)
Pitch: 0.50 m		(0.55 m pitch for section A - D)

Secondary moment incurred by prestressing

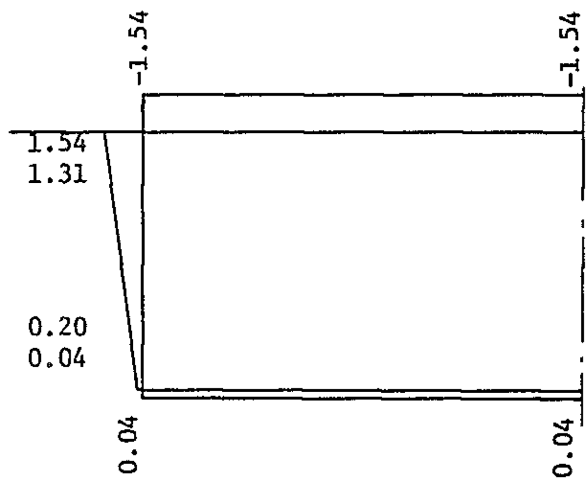


Fig. 5-39

h) Stress

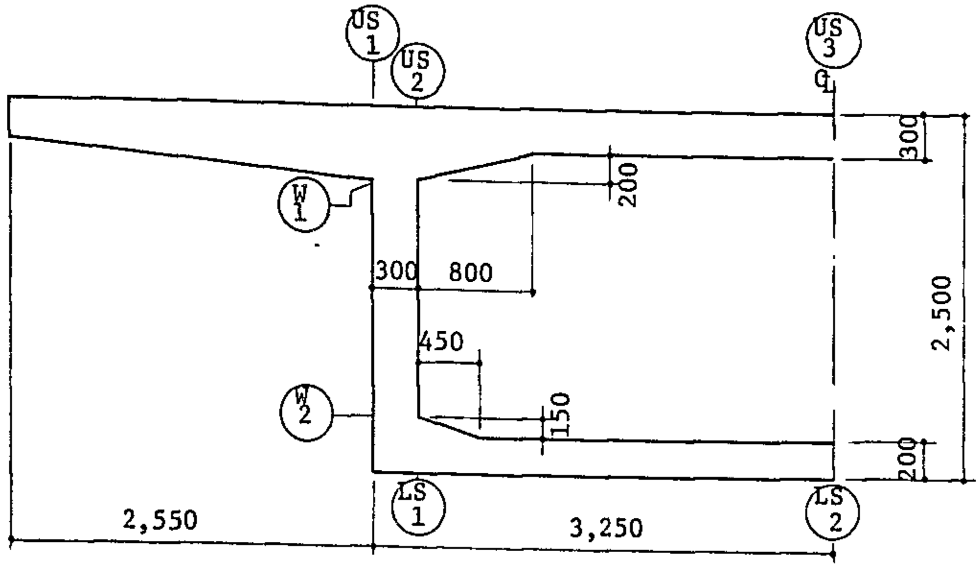


Fig. 5-40

SUM OF BENDING STRESS.

◆ UPPER SLAB(PC) (T/M2) NP= 96.40 (T/M) Effective prestress per m.

		THICK	G	T(+)	T(-)	NP/A	MP/Z		P-E	P-T	G+PT	MAX	MIN
US 2	1 2	0.500	-66	-101	-333	193	264	upper edge	457	537	471	356	123
								lower edge	-71	-84	-18	30	262
US 3	1 3	0.300	38	449	1	321	-167	upper edge	155	182	220	604	156
								lower edge	488	574	536	39	487

◆ WEB ......(RC)

		D CM (Effective height)	SSA KG/CM2	MOMENT TM/M	X CM (Effective prestrers)	SC KG/CM2 (Prestress)	AR CM2/M (reinforcement required) (Prestress applied)	DIA (diamter of reinforcing bar)	PITCH MM	AS CM2/M (Combined stress)
W 1	1 2	25.0	1800	4.907	7.9	55.5	12.19	D-19	∂200	14.32
		26.0		-5.600	8.4	57.4	13.41	D-19	∂200	14.32
W 2	1 3	25.0	1800	-0.824	◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆			D-◆◆	∂◆◆◆◆◆◆◆◆◆◆	
		26.0		-2.277	5.7	33.4	5.25	D-13	∂200	6.34

◆ LOWER SLAB (RC)

		D	SSA	MOMENT	X	SC	AR	DIA	PITCH	AS
LS 1	1 2	36.0	1800	-2.170	5.7	22.4	3.53	D-13	∂250	5.07
		36.0		-0.743	◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆			D-◆◆	∂◆◆◆◆◆◆◆◆◆◆	
LS 2	1 3	21.0	1800	0.851	◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆◆			D-◆◆	∂◆◆◆◆◆◆◆◆◆◆	
		21.0		2.720	6.0	47.8	7.95	D-16	∂200	9.93

* The effective height has been calculated taking the covering an external and internal surfaces of the box at 5.0cm and 4.0cm respectively.

Hangover Slab US 1

BENDING STRESS.

DANMEN-SEKI A= 0.500 (M2) DANMEN-KEISU Z= 0.041667 (M3)

	G	S	D=G+S	L	L(S)		TOTAL	TOTAL(S)
MOMENT	-2.818	-4.039	-6.857	-6.458	-12.169	*	-13.315	-19.026
STRESS	-67.6	-96.9	-164.6	-155.0	-292.1	*	-319.6	-456.6

* PRESTRESS

PE= 48.20 (T/EACH) PITCH= 0.500 (M) EX= 0.500/2-0.120 =0.130 (M)
amount of eccentricity

NP= PE/PITCH = 96.40 (T/M) MP=NP * EX= 12.532 (TM/M)

		UPPER FIBER	LOWER FIBER
STRESS OF PRESTRESS	NP/A	192.8	192.8
	MP/Z	300.8	-300.8
		* * * * * *	* * * * * * *
EFFECTIVE (effective prestress)	EPS	493.6	-108.0
INITIAL(EPS/0.85) (prestress applied)	IPS	580.7	-127.0

* TOTAL STRESS (T/M2).
Combined stress

		UPPER FIBER	LOWER FIBER
INITIAL	(G+IPS) on application	513.0	-59.4
LIVE MIN.	(D+EPS) under design condition	329.0	56.6
LIVE MAX.	(D+EPS+L) under design condition	174.0	211.6
SHOTOTSU	(D+EPS+LS) under impact load	37.0	348.6

Study of the edge of the hangover slab in the exial direction
LONGITUDINAL REINFORCEMENT.

M = (0.15L +0.13)*P.

JO-JI L= 0.500 T=0.455 P= 9.600

M= 1.968 (TM/M) D= 40.5 (CM) SS= 1400 (KG/CM2)

AS= 3.66 (D13 @250 A= 5.07) (CM2/M)

X= 6.1 (CM) SC= 16.69 (KG/CM2)

SHOTOTSU-JI L= 1.850 T=0.333 P= 9.600

M= 3.912 (TM/M) D= 28.3 (CM) SS= 2700 (KG/CM2)

AS= 5.51 (D13 @200 A= 6.34) (CM2/M)

X= 6.1 (CM) SC= 49.07 (KG/CM2)

6. Analysis of Stress in the Structure under Construction

In balanced moment in the structure under construction reaches the maximum when construction proceeds to the extent illustrated in Fig. 5-41. Safety of the substructure at this stage of construction is examined in this sub-chapter.

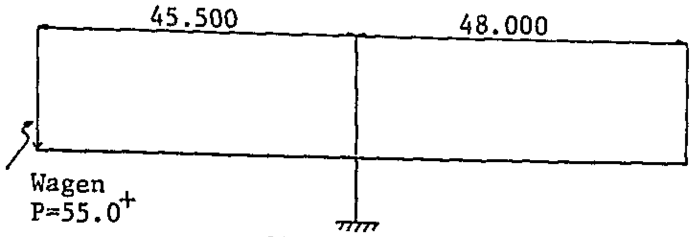


Fig. 5-41

Bending moment under dead load of the main girder at the above state (calculated by computer) is as in the following figure 5-42.

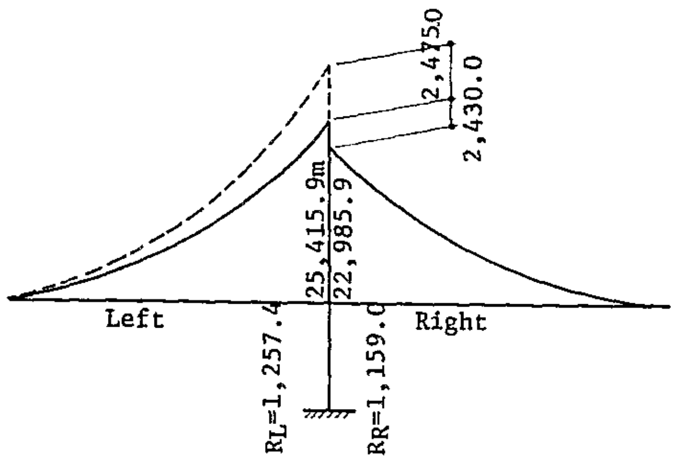


Fig. 5-42

(1) Pier Head

Under normal condition

$M = 2,430.0 + 2,475.0 = 4,905.0^{tm}$

$N = (1,257.4 + 1,159.0 + \underset{\text{Wagen}}{55} + \underset{\text{Cross beam}}{24.3} + \underset{\text{Column head}}{153.7}) = 2,649.4^{t}$

Calculation of the location of gravity center

(KV = 0.05 acting point)

$$\text{LEFT} = \frac{25,415.9 + 2,475.0}{1,257.4 + 55} = 21,252 \text{ m}$$

$$\text{RIGHT} = \frac{22,985.9}{1,159.0} = 19,833 \text{ m}$$

Under siesmic load

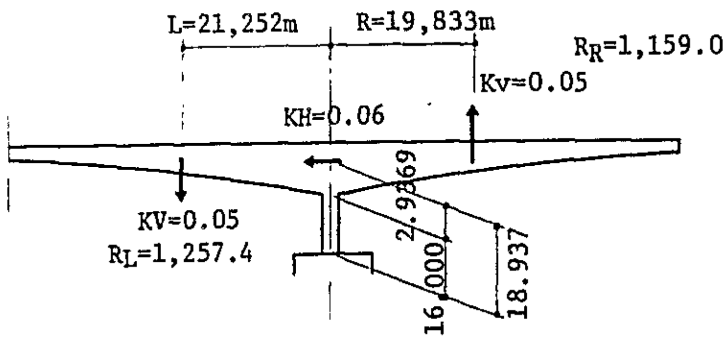


Fig. 5-43

$$M = (1,257.4 + 55) \times 0.05 \times 21,252 +$$
$$(1,257.4 + 1,159.0 + 153.7) \times 0.06 \times 2,936$$
$$(1,159.0 \times 0.05 \times 19,833) + 2,430 = 5,426.6^{tm}$$

$N = 2,649.4$

$H = 2,649.4 \times 0.06 \qquad = 159.0^{t}$

(2) Pier Foot

Under normal conditions

$M = 4,905.0$

$N = 2,649.4 + (18.75 \times 16.0 \times 2.5) = 3,399.4^{t}$ (m^2)

Under seismic load

$$M = 1,394.6 + (1,257.4 + 1,159.0 + 153.7) \times 0.06 \times 18,937$$
$$+ 1,149.3 + 2,430 + (750.0 \times 0.06 \times 16.0/2)$$

bridge pier $= 8,254.0^{tm}$

$N = 3,399.4^{t}$

Chapter 6. DESIGN OF SIMPLY SUPPORTED P.C. COMPOSITE BEAM

1. General

(1) The simply supported P.C. composite beam will be constructed by first placing precast P.C. beams (main beams) then combinning this with cross beams and R.C. slab to form a composite structure.

(2) The works will be executed by applying the Freyssinet Method, and for the P.C. steel material, P.C. steel wire 12-ø7mm will be used both in the main beam and in the cross beam.

(3) A study of the effective section of the main beam, making references to the "Specification for Road Bridge" of Japan Road Association, and "Specification for the Design of Prestressed Concrete Road Bridges" etc., reveals that all crosssections are fully effective over the entire span.

(4) In this design, internal forces in response to such dead loads as the dead weights of the main beam, the cross beam and the slab are calculated as those of a simple beam, while distribution of other types of dead load, static loads and live loads are calculated based on the GYON MOSSONET Distribution Method.

(5) The design load are taken as T-20 for the slab and L = 20 for both main and cross beams.

(6) In design calculation, it is necessary that stresses during each stage of construction be estimated in the sequence of work process. In this design, analysis of such stresses will be done based on the following concept of work process.

Concreting the main beam → Stressing and grouting : 90 days

Erection, cross beam, slab concreting (in-site) : 30 days

Surfacing, railing, paving (bridge surface) Live load, etc. : ∞

(7) The degree of creep and drying shrinkage in relation with the material age will be estimated according to the "Road Bridge Specification and Explanatory Notes" of Japan Road Association.

(8) In calculating various values pertaining to the properties of sections, converted equivalent values will be used for the strength of concrete which differs between the main beam and the slab.

(9) Design for the prevention of skidding between the main beam and the slab will be done in accordance with the "Road Bridge Specification and Explanatory Notes" of Japan Road Association.

(10) Cross fall of the bridge surface will be produced by the change in pavement thickness.

(11) This design will be done using the "Road Bridge Specification and Explanatory Notes" of Japan Road Association (Jan. 1978) as the basic guide line making reference also to other relevant specifications where necessary.

(12) Specifications used in this design are as follows:

- ° Road Bridge Specification and Explanatory Notes (Japan Road Association)
- ° Standard Specification for Reinforced Concrete (Japan Civil Engineering Society)
- ° Design and Construction Manual for Prestressed Concrete (Japan Civil Engineering Society)
- ° Design Manual - Vol. 2 (Japan Road Corporation)
- ° Specification for Prestressed Concrete Road Bridge and Explanatory Notes (Japan Road Association)
- ° Manual for the Design (and Construction) of Prestressed Concrete Composite Beam (Draft) (Japan Expressway Research Committee)
- ° Freyssinet Method Construction Standard (Freyssinet Conference)

° Standard Beam Layout and Standard Design of Slab
(Japan Road Corporation)

(13) <u>Design Process</u> (Design of the Main Beam)

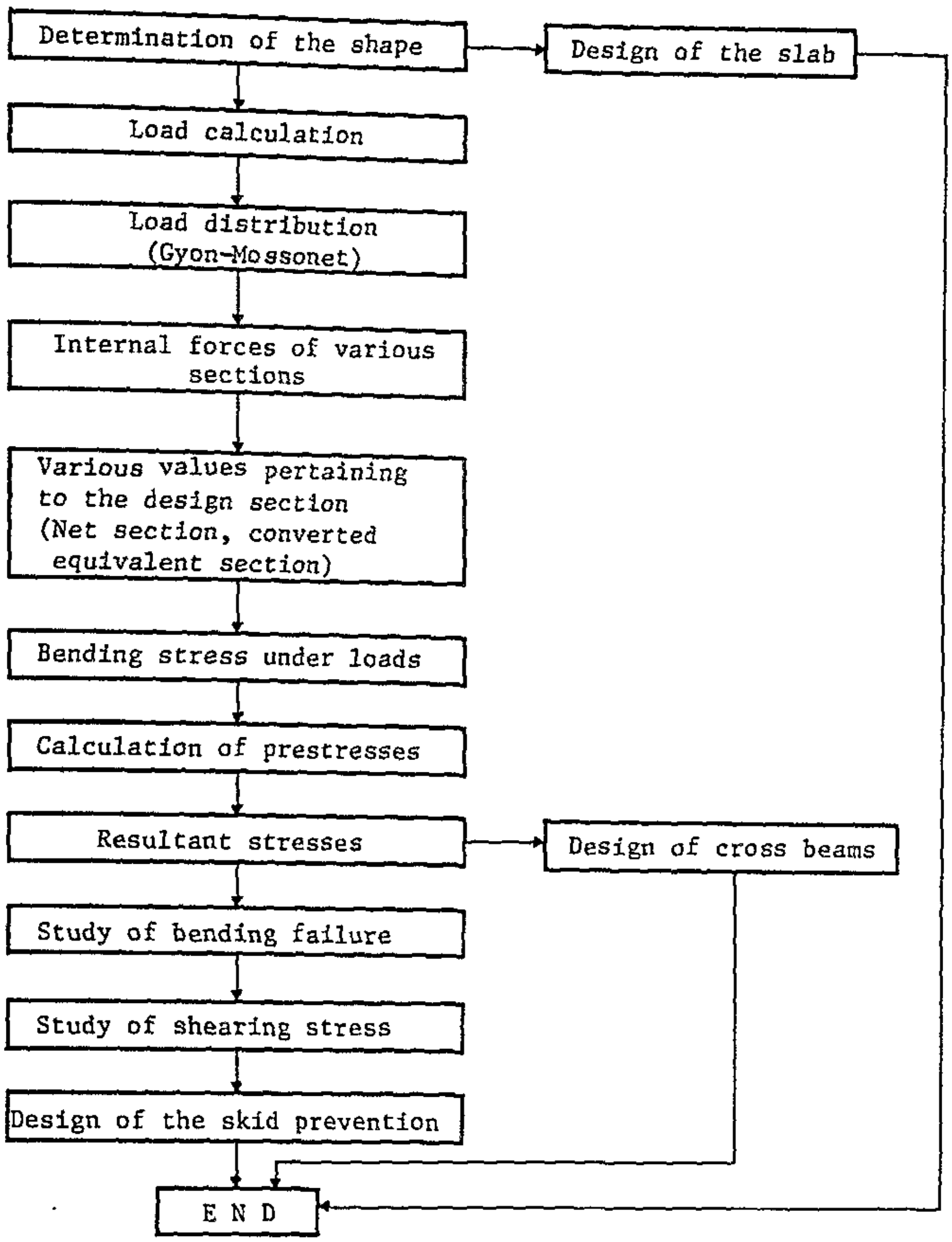


Fig. 6-1

2. Design Conditions

(1) Structural Features and Design Loads

Category of bridge	Prestressed Concrete Road Bridge
Form of structure	Post-tension P.C. composite beam
Length of bridge	30.000 m
Length of beam	29.920 m
Span	29.200 m
Effective width	1.500m + 8.000m + 1.500m
Oblique angle	LR
Cross fall	2.0%
Slope	LEVEL
Line load	TL-20
Impact coefficient	For T load $i = \frac{20}{50 + \ell}$
	For L load $i = \frac{10}{25 + \ell}$
Seismic coefficient	Horizontal seismic coefficient $K_H = 0$
	Verticle seismic coefficient $K_V = 0$
Safety against failure	
Ultimate load	1.3 x (Dead load) + 2.5 x (Live load x Impact load)
	1.7 x (Dead load + Live load x Impact load)
Prestress	Partial prestress

(2) Strength of Material and Allowable Stress

a) Concrete

			Cross beam Precast beam	Slab
Design standard strength		σCK	= 350kg/cm^2	240kg/cm^2
Allowable bending compressive stress	Immediately after prestressing	σcat	= 160 "	——
	Others	σpa	= 125 "	68.5 "
Allowable bending tensile stress	Immediately after prestressing	$\sigma cat'$	= 13.5 "	——
	Live load main loads other than impact load	$\sigma ca'$	= 0 "	——
	Others	$\sigma ca'$	= 13.5 "	——
Allowable shearing stress	Under design load	τa	= 5.0 "	3.9 "
	Maximum value	τa max	= 46.5 "	32 "
Allowable principal tensile stress	Shearing force alone	σI	= 9 "	——
Compressive force during prestressing operation		σci	= 290 "	——

b) P.C. Steel Material

			P.C. steel wire 12 - ϕ7	P.C. steel wire 12 - ϕ5
Tensile force		σpk	= 155kg/mm^2	165kg/mm^2
Stress at yielding point		σpy	= 135 "	145 "
Stress tensile	During stressing operation	σpsi	= 121.5 "	130.5 "
	Immediately after prestressing	σpst	= 108.5 "	115.5 "
	Under design stress	σpa	= 93.0 "	99.0 "

c) Reinforcing bar (SD30)

Allowable stress	Slab	sa	= 1,400kg/cm^2
	Main beam, cross beam	sa	= 1,800 "
	For calculating the required stirrup	sa	= 3,000 "

(3) Other Considerations

Relaxation of P.C. Steel Wire

Main Beam (main cable) cross beam (lateral bracing) γ = 5%

Creep modulus of concrete	(∞)	= 2.60
	(90)	= 1.70
	(120)	= 1.64
Drying shrinkage ratio of concrete	(∞)	= 20 x 10^{-5}
	(90)	= 16 "
	(120)	= 15.6 "

Young's modulus of concrete

Precast beam	During prestressing operation	E_{c1} = 3.0 x 10^5kg/cm^2
	Design standard	E_{c1} = 3.25 "
Cross beam	During prestressing operation	E_{c2} = 3.0 "
	Design standard	E_{c2} = 3.25 "
Slab	Design standard	E_{c3} = 2.7 "

Young's Modulus of P.C. Steel $\quad$ Ep = 20.0 x 10^5kg/cm^2

Frictional coefficient between P.C. steel material and sheath

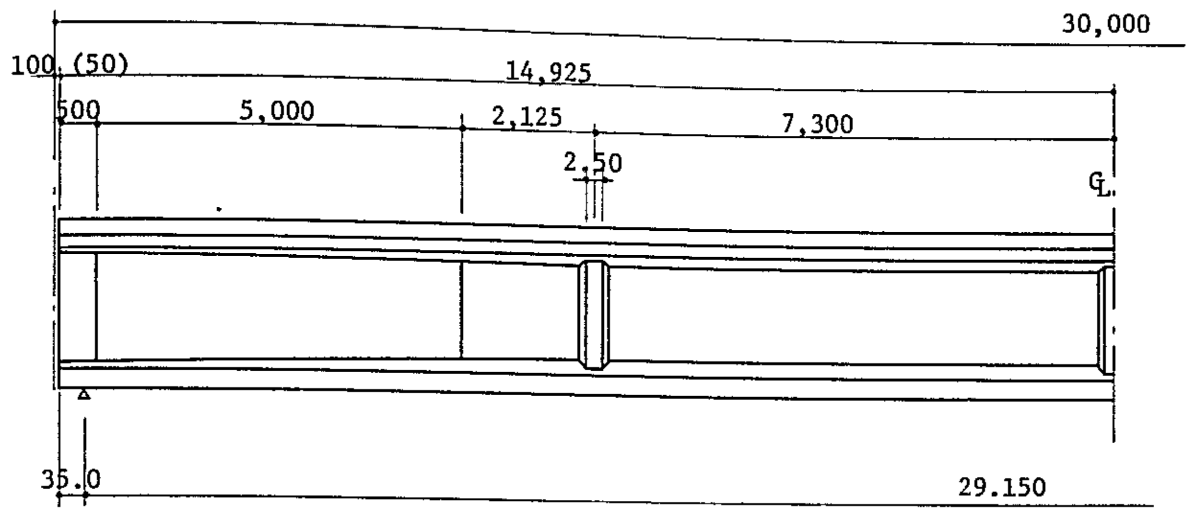
Per unit (1.0m) length = 0.004

Per radian = 0.3

Amount of setting	12 - ϕ7	5 mm
	12 - ϕ5	4 mm

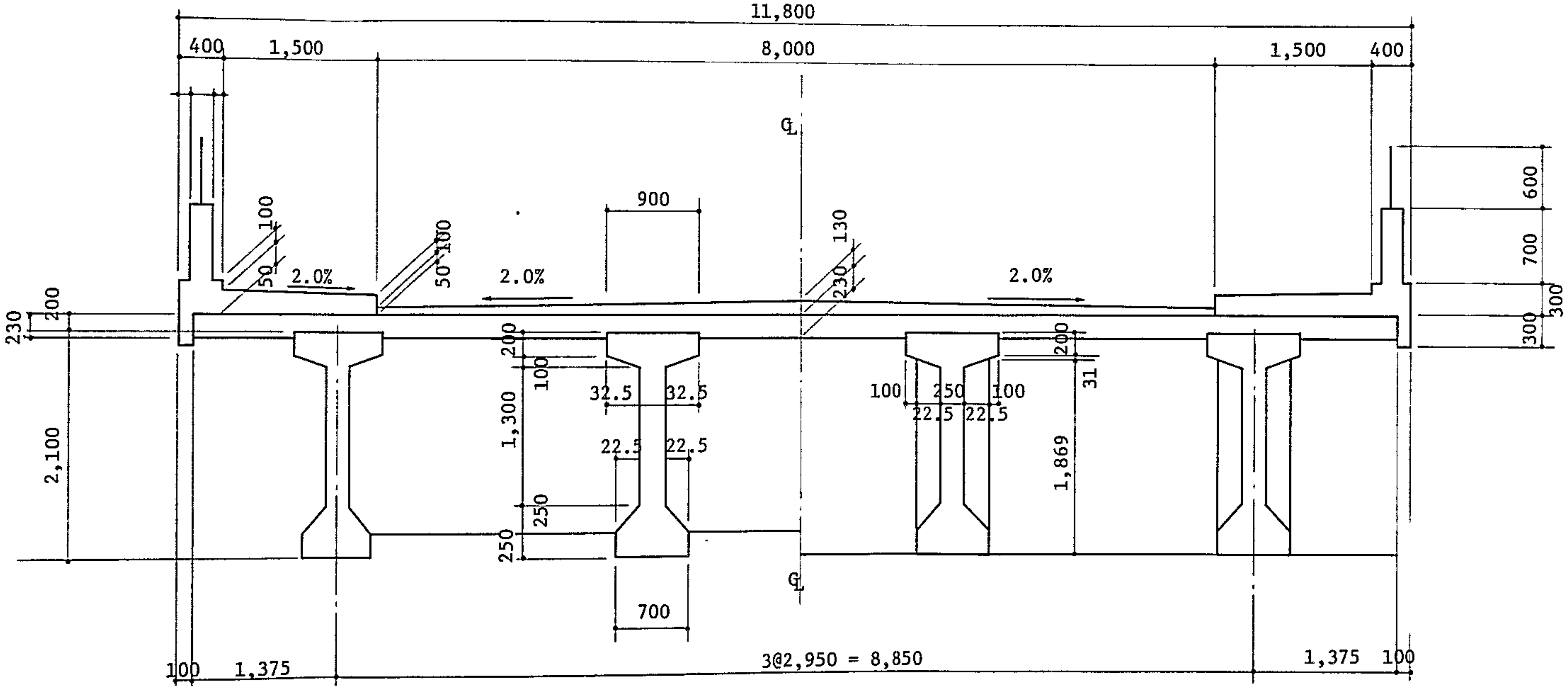
3. Dimension

Lateral View of Main Beam



Mid-span Section

Section at support



4. Design Calculation

(1) Load Calculation

Static load

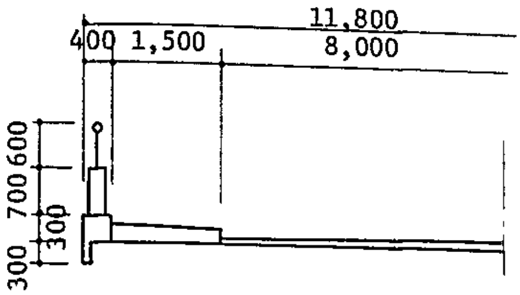


Fig. 6-2

Surfacing, railing (One side)

W_1 = (0.20x0.70+0.40x0.30+0.10x0.30) x 2.5 + 0.05 = 0.775 t/m

Pavement

W_2 = 1/2 x (0.05+0.13) x 2.3 = 0.207 t/m^2

Sidewalk

W_3 = 1/2 x (0.20+0.17) x 2.5 = 0.463 t/m^2

Appurtenants (Water pipes)

W_y = 0.300 t/m

Line load (TL-20)

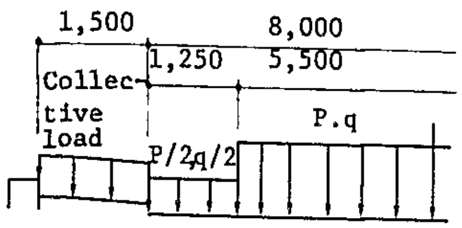


Fig. 6-3

Uniform load ($q = 0.35\ t/m^2$)

$$W_q = 0.35 \times \left\{5.5 + \frac{8.0-5.5}{2}\right\} = 2.363\ t/m$$

Linear load P = 5.00 t

$$\Sigma P = 5.00 \times \left\{5.5 + \frac{8.00-5.5}{2}\right\} = 33.75\ t$$

Collective load W' = 0.35 t/m^2

W = 0.35 x 1.5 x 2 = 1.05 t/m

Impact coefficient: i

$$i = \frac{10}{25 + \ell} = \frac{10}{25 + 29.2} = 0.185$$

(2) Design Internal Forces (Bending Moment and Shearing Force)

Based on the GYON MOSSONET Theory of Distribution, the sectional forces of a selected section of the beam are obtained as follows:

Bending Moment (Center of span)

	Beam G_1	Beam G_2	
Dead load of main beam	248.3	248.3	
Concrete between main beams	185.9	210.6	Slab Cross beams
Bridge surface loads	148.4	135.8	
Live loads	211.5	185.6	Live load Collective loads
Total	794.1	780.3	

Shearing Force (at point of support)

	Beam G_1	Beam G_2	
Dead load of main beams	36.8	36.8	
Concrete between main beams	24.8	27.6	Slab Cross beams
Bridge surface loads	20.3	18.6	
Live loads	(8.9) 29.0	(8.1) 25.4	Live loads Collective loads
Total	(8.9) 110.9	(8.1) 108.4	

Figures in ( ) show the value at center of span.

Reaction (at point of support)

	Beam G_1	Beam G_2	
Dead load of main beam	38.1	38.1	
Concrete between main beams	28.0	33.3	Slab Cross beams
Bridge surface loads	20.8	19.1	
Live loads	29.4	25.8	Live loads Collective loads
Total	116.4	116.3	

Using the above data design of the Beam G_1 will be studied as a representing case, the result of which will be applied also to other beams.

To determine the internal force of various sections on Beam G_1, it is assumed that the magnitude of bending moment varies continuously along the beam in a parabolic curve, and that the shearing force distributes along the beam in straight lines.

Bending moment

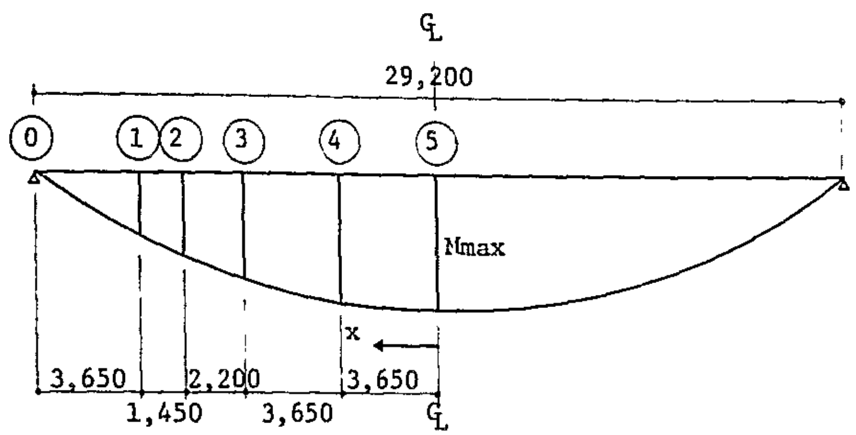


Fig. 6-4

$$M_x = M_{max}\left\{1 - \left(\frac{X}{\ell}\right)^2\right\}$$

	1	2	3	4	5
$\left\{1-\left(\frac{X}{\ell}\right)^2\right\}$	0.430	0.577	0.750	0.938	1.000
Dead load of main beam	108.8	143.3	186.2	232.9	248.3
Concrete between main beams	81.4	107.3	139.4	174.4	185.9
Bridge surface load	65.0	85.6	111.3	139.2	148.4
Life loads	92.6	122.0	158.6	198.4	211.5
Total	347.8	458.2	595.5	744.9	794.1

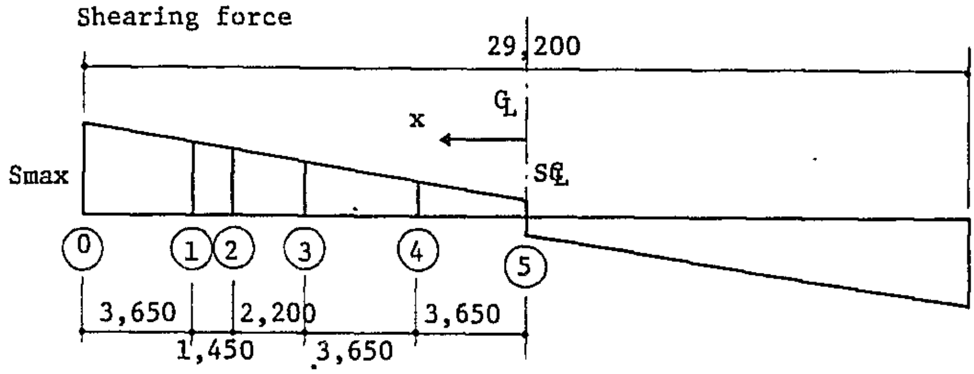


Fig. 6-5

$$S_X = \frac{X}{\ell}(S_{max} - S℄) + S℄$$

	0 (Smax)	1	2	3	4	5 (S℄)
X/ℓ	1.000	0.750	0.651	0.500	0.250	——
Dead load of main beam	36.8	27.6	24.0	18.4	9.2	0
Concrete between main beams	24.8	18.6	16.1	12.4	6.2	0
Bridge surface load	20.3	15.2	13.2	10.2	5.1	0
Live load	29.0	24.0	22.0	18.9	13.9	8.9
Total	110.9	85.4	75.3	59.9	34.4	8.9

(3) Calculation of Constants Relevant to the Design Sections

The relevant constants will be obtained by computer as the solutions of formulas shown in the following table.

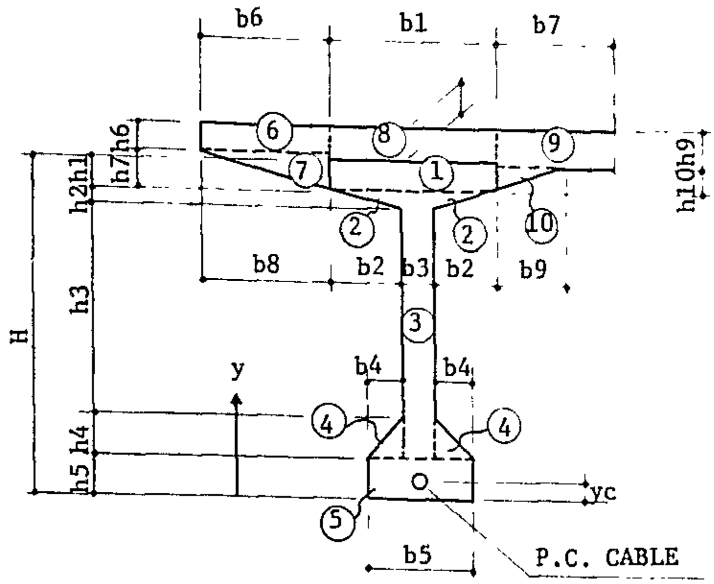


Fig. 6-6

	A (cm^2)	Y (cm)	$A{\cdot}Y\,(cm^3)$	$A{\cdot}Y^2\,(cm^4)$	I' (cm^4)
1	$b1{\cdot}h1$	$H-h/2$	$A_1{\cdot}Y_1$	$A_1{\cdot}Y_1^2$	$1/12{\cdot}A_1{\cdot}h_1^2$
2	$b_2{\cdot}h_2$	$H-(h_1+h_2/3)$	.	.	$1/18.A_2.h_2^2$
3	$b_3(h_2+h_3+h_4)$	$1/2(h_2+h_3+h_4)+h_5$	.	.	$1/12(h_2+h_3+_4)^2{\cdot}A_3$
4	$b_4.h_4$	$\frac{h}{3}+h_5$	.	.	$1/18.A_4.h_4^2$
5	$b_5{\cdot}h_5$	$\frac{h5}{2}$	.	.	$1/12.A_5.h_5^2$
	$\sum_{1}^{5}.A$		$\sum_{1}^{5}.A.Y$	$_{1}^{5}.A.Y^2$	$\sum_{1}^{5}.I'$
Total section	$Y = -\Sigma.A.Y/\Sigma A$ $Y' = H + Y$ $I = \Sigma A.Y^2 + \Sigma I' - \Sigma A.Y^2$ $W' = I/Y'$ $W = I/Y$ $R^2 = I/\Sigma A$				

(4) Example of calculation Section 5

	b x h = A	Y	A·Y	$A \cdot Y^2$	I'
①	90.0x20.0 = 1,800	200.0	360,000	72,000,000	60,000
②	32.5x10.0 = 325	186.7	60,666.7	11,324,444.4	1,805.6
③	25.0x165.0= 4,125	107.5	443,437.5	47,669,531.3	9,358,593.7
④	22.5x25.0 = 562.5	33.3	18,750	625,000	19,531.2
⑤	70.0x25.0 = 1,750	12.5	21,875	273,437.5	91,145.8
	8,562.5		904,729.2	131,892,413.2	9,531,076.3

Total section

$$Y = \frac{-904,729.2}{8,562.5} = -105.7\text{cm} \qquad Y' = 210 - 105.7 = 104.3\text{cm}$$

$$I = 131,892,413.2 + 9,531,076.3 - 8,562.5 \times 105.7^2$$
$$= 45,828,176.8\ \text{cm}^4$$

$$W' = \frac{45,828,176.8}{-105.7} \approx -433,725.1\text{cm}^3$$

$$W = \frac{45,828,176.8}{104.3} \approx 439,227.2\text{cm}^3$$

$$R^2 = \frac{45,828,176.8}{8,562.5} \approx 5,352.8\ \text{cm}^2$$

Sheath hole	-13x15,904 = -206.8	17.7	-3,659.5	-64,773.3	———
	8,355.7		901,069.7	131,827,639.9	9,531,076.3

Net section

$$Y = \frac{901,069.7}{8,355.7} \approx -107.8 \qquad Y' = 210.0 - 107.8 = 102.2\text{cm}$$

$$Epc = -107.8 + 17.7 = -90.1\text{cm}$$

Other required calculation can be done following the above example.

$$\delta = \frac{M}{W}$$

Load	Bending moment	Section modulus	Upper edge of slab	Upper edge of main beam	Lower edge of main beam	Location of centroid of steel wire
[illegible]	[illegible] (t·m)	[illegible] (10+5cm+3)	δ 1	δ 2 (1.3 cm+2)	δ 3	δ 4
Main beam	248.3	Wc' = 4.325	---	57.4	---	---
		Wc = -4.098	---	---	-60.6	---
		Wc3 = -4.902	---	---	---	-50.7
Slab cross beam	185.9	We1 = 4.441	---	41.9	---	---
		We = -4.524	---	---	-41.1	---
		We31= -5.452	---	---	---	-34.1
Bridge surface	148.4	We2"= 10.900	13.6	---	---	---
		We2 = 14.363	---	10.3	---	---
		We2 = -6.149	---	---	-24.1	---
		We32= -6.990	---	---	---	-21.2
Live load	211.5	We2"= 10.900	19.4	---	---	---
		We2'= 14.363	---	14.7	---	---
		We2 = -6.149	---	---	-34.4	---
		We32= -6.990	---	---	---	-30.3

(5) Prestressing

Prestressing will be applied, by using 13 pieces of P.C. steel wire of 12 - ϕ7mm size.

a) Stresses in the P.C. Wire immediately after Prestressing

i) Stress of the P.C. Steel Wire immediately after Prestressing

Assume the stress of the steel wires immediately before anchoring to be $\sigma p'i = 112\ kg/mm^2$. The stress σ_{pt} of P.C. steel wires in the design section decreases immediately after prestressing due to friction between the steel wire and the sheath, slipping of the anchorage, and elastic deformation of the main beam concrete.

ii) Decrement due to friction between the steel wires and the sheath

$$pt' = Pi \cdot e^{-(\mu\alpha + \lambda\ell)}$$

Where,

μ : Frictional coefficient per radian of angular change = 0.3

α : Angle of the bent up cable (in radians)

λ : Frictional coefficient per unit length (1.0m) = 0.004

ℓ : Length of the steel wire between the design section and the anchorage element

σ_{pt}' at the design section 5 :

$$\sigma_{pt}' = 101.7\ kg/mm^2$$

iii) Decrement of Stress of the P.C. Steel Wires due to Elastic Deformation

$$\sigma P2 = 1/2 \cdot n\ (\sigma ctg + \sigma dog)\ \frac{N-1}{N}$$

Where,

n; Ratio of Young's modulus; $Ep/Ec = 20x10^5/3.0x10^5 = 6.667$

$$\sigma ctg = \frac{Pt'}{A} + \frac{Pt' \cdot epc}{Wcg} = 185.3\ kg/cm^2$$

$$Pt' = N \cdot Ap \cdot \sigma pt' = 610.5$$

$$\sigma dog = \frac{Mdo}{Wig} = -50.7\ kg/cm^2$$

Therefore the stress of P.C. steel wires in the design section immediately after prestressing is,

$$\sigma pt = \sigma pt' - \sigma p2 = 101.7 - 4.1 = 97.6 \text{ kg/cm}^2$$

$$Pt = N \cdot Ap \cdot \sigma pt = 13 \times 461.8 \times 97.6 = 585.5 \text{ t}$$

and the stress in the section of the beam incurred by prestressing is.

$$Pt = N \cdot Ap \cdot \sigma pt$$

$$= 13 \times 461.8 \times 97.6 = 585.9 \text{ t}$$

$$Mpt = Pt \cdot epc = 585.9 \times (-0.901) = -527.9 \text{ t.m}$$

Stress at upper edge of the beam,

$$\sigma pt' = \frac{585.9 \times 10^3}{8.356 \times 10^3} + \frac{-527.9 \times 10^5}{4.325 \times 10^5} = -51.9 \text{ kg/cm}^2$$

Stress of lower edge of the beam

$$\sigma pt = 70.1 + \frac{-527.9 \times 10^5}{-4.098 \times 10^5} = 198.9 \text{ kg/cm}^2$$

Stress at P.C.W. centroid

$$\sigma dg = 70.1 + \frac{-527.9 \times 10^5}{-4.902 \times 10^5} = 177.8 \text{ kg/cm}^2$$

b) Effective Prestress

The prestress applied to the concrete decreases due to creep and drying shrinkage of the concrete and relaxation of the P.C. steel wire.

i) Decrement of stress of the P.C. steel wires due to creep and drying shrinkage of concrete.

$$\sigma p\psi = \frac{n \cdot \phi 1(\sigma ctg + \sigma dog + Ep \cdot \varepsilon s)}{1 + \alpha\left(1 + \frac{\phi 1}{2}\right)} + \frac{n \cdot \phi 2(\sigma d1g + \sigma d2g)}{1 + \alpha\left(1 + \frac{\phi 2}{2}\right)}$$

$$+ \frac{n.\phi 3.\sigma d3g}{1 + \alpha\left(1 + \frac{\phi 3}{2}\right)}$$

Where,

σctg: Stress of concrete at the centroid of the P.C. steel wires immediately after prestressing.

σdig: Bending stress at the centroid of the P.C. steel wires due to dead load.

σdog: Dead weight of main beam

$\sigma d_1 g$: Dead weight of cross beam

$\sigma d2g$: Dead weight of slab

$\sigma d3g$: Bridge surface load

n : Ratio of young's modulus of main beam concrete and P.C. steel wires

$$n = \frac{Ep}{Ec}$$

Ep : Young's modulus of P.C. steel wires

Ec : Young's modulus of main beam concrete

Es : Modulus of drying shrinkage of concrete

ϕ : Creep coefficient *

$\phi_1 = \phi\infty = 2.60$

$\phi_2 = \phi\infty - \phi\,(90) = 1.70$

$\phi_3 = \phi\infty - \phi\,(120) = 1.64$

* From "Prestress Concrete Road Bridge Specification", item 2.2.2.

α : $\alpha = n\,\frac{\sigma tg}{\sigma pt}$

σpt : Tensile stress of the P.C. steel wires immediately after prestressing.

ii) Decrement of the stress due to relaxation of the P.C. steel wires.

The increment is assumed at 5%.

$$\Delta\sigma p\gamma = 0.05 \cdot \sigma pt$$

iii) Results Efflective stress of the P.C. steel wire

$\sigma pe = \sigma pt - \Delta\sigma p\phi - \Delta\sigma p\gamma$

Effective coefficient: K

$$K = \frac{\sigma pe}{\sigma pt}$$

Effective prestress

At upper edge of beam $\sigma ce' = K \cdot \sigma ct'$

At lower edge of beam $\sigma ce = K \cdot \sigma ct$

iv) Example of calculation for effective prestress and effective coefficient of design section 5.

Effective prestress

$\sigma pe = \sigma pt - \sigma p\phi - \sigma p\gamma$

$p\gamma = 0.05 \times \sigma pt = 0.05 \times 97.6 = 4.9\ kg/mm^2$

$\sigma pe = 97.6 - 15.6 - 4.9 = 77.1\ kg/mm^2$

Effective coefficient

$$K = \frac{\sigma pe}{\sigma pt} = \frac{77.1}{97.6} = 0.790$$

Stress under effective prestress

$$\sigma ce = \sigma ct \cdot K$$

Section	σpt	σpϕ	σpr	σpe	σK
0	94.5	10.0	4.7	79.8	0.844
1	95.7	13.1	4.8	77.8	0.813
2	96.2	15.0	4.8	76.4	0.794
3	97.3	15.7	4.9	76.7	0.789
4	98.6	16.5	4.9	77.2	0.783
5	97.6	15.6	4.9	77.1	0.790

	Stress immediately after prestressing				Effective prestress		
Section	σct'	σct	σctg	K	σce'	σce	σceg
0	36.3	39.8	38.0	0.844	30.7	33.6	32.1
1	3.0	113.7	80.1	0.813	2.5	92.4	65.1
2	-5.9	147.2	110.7	0.794	-4.7	116.8	87.9
3	-27.8	172.2	139.5	0.789	-21.9	135.9	110.0
4	-51.3	199.8	177.7	0.783	-40.2	156.5	139.1
5	-51.9	198.9	177.7	0.790	-41.0	157.1	140.4

(6) Stress under Other Loads

a) Stress due to the difference of concrete age between the main beam and the slab

Difference of concrete age between the main beam and the slab results in the difference of drying shrinkage and creep, which leads to the development of internal stress. Calculation of such internal stress is done as follows:

i) Internal stress developed from differential drying shrinkage

The relation between the rate of drying shrinkage and time factor can be formulated as:

$$S(t) = \frac{S(\infty)}{\psi(\infty)} \cdot \psi(t)$$

Where,

$S(t)$: The rate of drying shrinkage developed during any amount of time t (in days)

$S(\infty)$: Ultimate rate of drying shrinkage: 20×10^{-5}

$\psi(\infty)$: Ultimate creep coefficient: 2.6

$\psi(t)$: Creep coefficient during time t.

b) Stress due to the difference of temperature between the main beam and the slab

The exposure of the slab to the sun causes the difference of temperature between the slab, and the main beam, and results in the development of internal stress in the structure.

The stress thus developed will be calculated assuming such temperature difference to be 5°C.

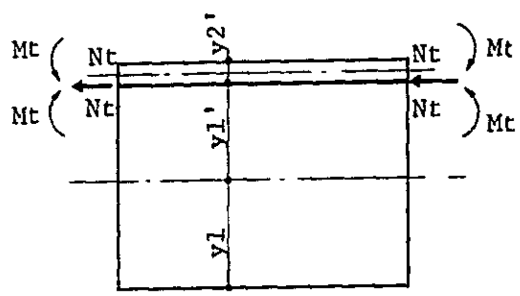


Fig. 6-7

(7) Resultant Stress

Section 5

		Immediately after prestressing		Under design load (kg/cm^2)		
		Upper edge of beam	Lower edge of beam	Upper edge of slab	Upper edge of beam	Lower edge of beam
①	Stress immediately after prestressing	-51.9	198.9			
②	Effective prestress				-41.0	157.1
③	Dead load of main beam	57.4	-60.6		57.4	-60.6
④	Cross beam, slab				41.9	-41.1
⑤	Bridge surface load			13.6	10.3	-24.1
⑥	Live load			19.4	14.7	-34.4
⑦	Differential drying shrinkage			-0.8	2.3	-0.9
⑧	Differential creep coefficient			9.3	-25.0	10.3
⑨	Temperature difference			3.5	-10.4	4.3
Resultant stress						
①+③		5.5	138.3			
Allowable value		$\sigma_{cat}=-13.5$	$\sigma_{cat}=160$			
②+③+④+⑤+⑥+⑦+⑧				41.5	60.6	6.3
Allowable value				$\sigma''_{ca}=80$	$\sigma'_{ca}=125$	$\sigma_{ca}=-13.5$
②+③+④+⑤+⑥+⑦+⑧ + 9				45.0	50.2	10.6
Allowable value				$\sigma''_{ca}=92$	$\sigma'_{ca}=144$	$\sigma_{ca}=-18.5$
At 70% of difference in drying shrinkage and creep coefficient						
⑦'	Difference in drying shrinkage			-0.6	1.6	-0.6
⑧'	Difference in creep coefficient			6.5	-17.5	7.2
Resultant stress						
②+③+④+⑤+⑥+⑦'+⑧'				38.9	67.4	3.5
Allowable value				$\sigma''_{ca}=80$	$\sigma'_{ca}=125$	$\sigma_{ca}=-13.5$
②+③+④+⑤+⑥+⑦'+⑧' + 9				42.4	57.0	7.8
Allowable value				$\sigma''_{ca}=92$	$\sigma'_{ca}=144$	$\sigma_{ca}=-18.5$

(8) Stress of the P.C. Steel Wires   Section 5

The tensile stress of the P.C. steel wires increases while the load increases.

The increment of tensile stress in the P.C. steel wire is calculated as in the following:

$$\Delta\sigma p = \eta \cdot (\sigma d_1 g + \sigma d2g + \sigma eg)$$

Where,

$$\eta = \frac{Ep}{Ec} = \frac{20 \times 10^5 \text{ kg/cm}^2}{3.25 \times 10^5 \text{ kg/cm}^2} = 6.154$$

$$\Delta\sigma p = \eta \cdot (34.1 + 21.2 + 30.3)$$
$$= 6.154 \times 85.6 = 527 \text{ kg/cm}^2$$
$$\fallingdotseq 5.3 \text{ kg/mm}^2$$

Maximum tensile stress of the P.C. steel wire is determined as follows:

$$\sigma p \max = \sigma pe + \Delta\sigma p$$

$$\sigma p \max = 77.1 + 5.3 = 82.4 \text{ kg/mm}^2 < \sigma pa = 93.0 \text{ kg/mm}^2$$

(9) Design of Slab

a) Bending Moment

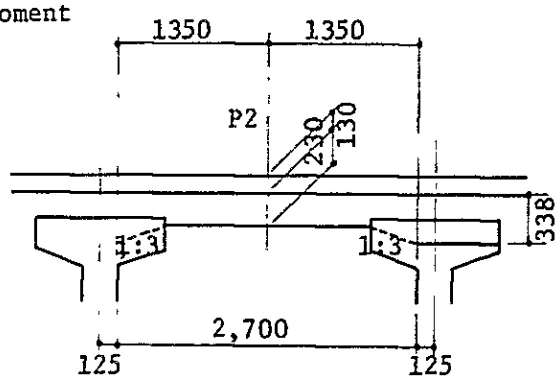


Fig. 6-8

Dead load

Slab	$W_{so} = 0.230 \times 2.5 =$	0.575 t/m
Pavement	$W_{po} = 0.130 \times 2.3 =$	0.299 "
	$W_d =$	0.874 t/m

Bending moment of the supporting point

Dead load

$$Md = -\frac{1}{10} \times 0.874 \times 2.700^2 = -0.637 \text{ t.m/m}$$

Live load

$$M\ell = -(0.15 \quad + 0.125) \cdot P$$
$$= -(0.15 \times 2.700 + 0.125) \times 9.6$$
$$= -5.088 \text{ t.m/m}$$

Total

$$\Sigma M = -0.637 - 5.088 = -5.725 \text{ t.m/m}$$

Bending moment at center of span

Dead load

$$Md = \frac{1}{10} \times 0.874 \times 2.700^2 = 0.637 \text{ t.m/m}$$

Live load

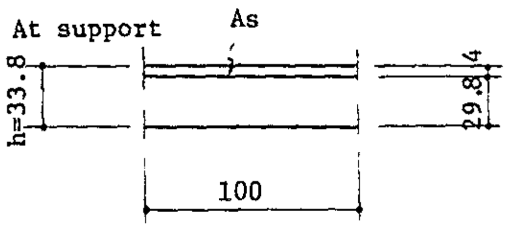
$$M\ell = (0.12 \cdot \ell + 0.07) \cdot P \cdot 0.8$$
$$= (0.12 \times 2.700 + 0.07) \times 9.6 \times 0.8$$
$$= 3.026 \text{ t.m/m}$$

Total

$$\Sigma M = 0.637 + \ 3.026 = 3.663 \text{ t.m/m}$$

b) Stress analysis

At support



M = 5.725 t.m

As = D16 - atc 125

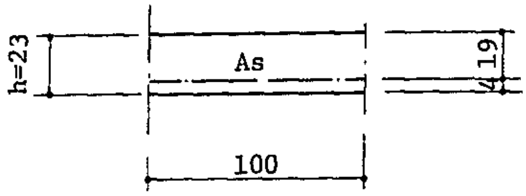
= 15.888 cm^2

Fig. 6-9

$$\sigma c = \frac{M}{b \cdot d^2} - \frac{1}{Lc} = 6.45 \times 6.85 = 44.1 kg/cm^2 < 68.5 kg/cm^2$$

$$\sigma s = \text{''} - \frac{1}{Ls} = \text{''} \times 210.6 = 1{,}358 \text{ ''} < 1{,}400 \text{ ''}$$

c) At center of span



M = 3.663 t.m

As = D16 - atc 125

$= 15.888\ cm^2$

Fig. 6-10

$$\sigma c = \frac{M}{b \cdot d^2} \times \frac{1}{Lc} = 10.15 \times 5.88 = 59.7\ kg/cm^2 < 68.5\ kg/cm^2$$

$$\sigma s = \text{"} \times \frac{1}{Ls} = \text{"} \times 137.5 = 1{,}395\ \text{"} < 1{,}400\ \text{"}$$

Chapter 7. DESIGN OF SUBSTRUCTURES

1. Design Condition

a) Cases involved: Substructures for piers A_1, A_2, P_1 -4

b) Types of Structure: Inverted T abutment, wall-type pier, T-type pier

c) Foundations:

Cast-in-situ concrete pile 1,500ϕ

Caisson foundation 14.00mϕ

d) Relevant Constants

Design Seismic Coefficient

Horizontal
- on completion $K_H = 0.12$
- during construction $K_H = 0.06$

Verticle $K_v = \pm 0$

Frictional coefficient at bearing portion for super structure
(Bearing plate will be used.) $u = 0.15$

Properties of Soil

Loaded soil and backing soil

Angle of internal friction $\phi = 30°$

Angle of friction between soil and surface of structure

Between soil and earthwork $\delta = \begin{cases} \phi \ldots \text{normally} \\ \frac{\phi}{2} \ldots \text{during earthquake} \end{cases}$

Between soil and concrete work $\delta = \begin{cases} \frac{\phi}{3} \ldots \text{normally} \\ 0 \ldots \text{during earthquake} \end{cases}$

Unit weight $\gamma = 1.8\ t/m^3$

Properties of bearing soil

Properties of bearing soil are to be determined based on foundation investigation data for each individual substructure.

Properties of Concrete

Young's modulus (Ec)

(kg/cm^2)

σck	180	210	240	300
Ec	2.4×10^5	2.55×10^5	2.7×10^5	3.0×10^5

Unit weight

Reinforced concrete $c = 2.5\ t/m^3$

Plane concrete $c = 2.35$ "

Other Constants

Ratio of Young's modulus of reinforcing bar to concrete $n = 15$

Poisson's ratio $= \frac{1}{6}$

Unit weight

Steel material $= 7.85\ t/m^3$

Asphalt $= 2.3\ t/m^3$

e) Allowable Strength of Material

Reinforcement (To be SD - 30 or higher in standard)

Tensile stress $\sigma sa = 1{,}800\ kg/cm^2$

(If constantly underwater or underground: $\sigma sa = 1{,}600\ kg/cm^2$)

Concrete

Design standard strength		kg/cm^2	ck=180	ck=210	ck=240	ck=300
Bending compressive stress		"	60	70	80	100
Shearing stress	Slab	"	8	8.5	9	10
	Beam		6	6.5	7	8
Bearing stress		"	54	63	72	90
Bond stress		"	14	15	16	18

Allowable tensile stress

Normal concrete $\sigma ca = -3\ kg/cm$

Underwater concrete (70% of normal concrete)

$\sigma ca = -2.1\ kg/cm^2$

Steel Material

Steel of SS-41 standard will be used.

Allowable bending compressive stress $ca = 1{,}400\ kg/cm^2$

Allowable bending tensile stress $ta = 1{,}400$ "

f) Stability of the Foundation

Stability in Vertical Bearing Strength

To be calculated based on soil investigation data for the individual site.

Horizontal Stability

Allowable displacement of piles and caisson foundation

Ground surface $\begin{cases} \text{Normally} & \delta a = 10\ mm \\ \text{Under earthquake} & \delta e = 20\ mm \end{cases}$

Allowable horizontal soil bearing strength of the caisson

Sandy soil $P_p = \gamma K_p \cdot \chi$

Cohesive soil $P_p = \gamma K_p \cdot \chi + 2C\sqrt{K_p}$

Under earthquake $P_p = (1 + k_v)\ \gamma \cdot \chi\ K_{EP} + 2C\ \sqrt{K_{EP}}$

P_p : Passive earth pressure at a depth χ

γ : Unit weight of soil

C : Cohesive force of soil

The allowable horizontal soil bearing strength will be taken at 20% in excess of the passive earth pressure thus obtained.

g) Buoyancy

Land Section

Only the influence of ground water on footings will be considered.

Underwater Section

The smaller the section of the column and the larger the axial force, the more unfavorable situation the stability will be in. Buoyancy will therefore be considered for caisson portions.

2. Design of Abutments

a) Dimension

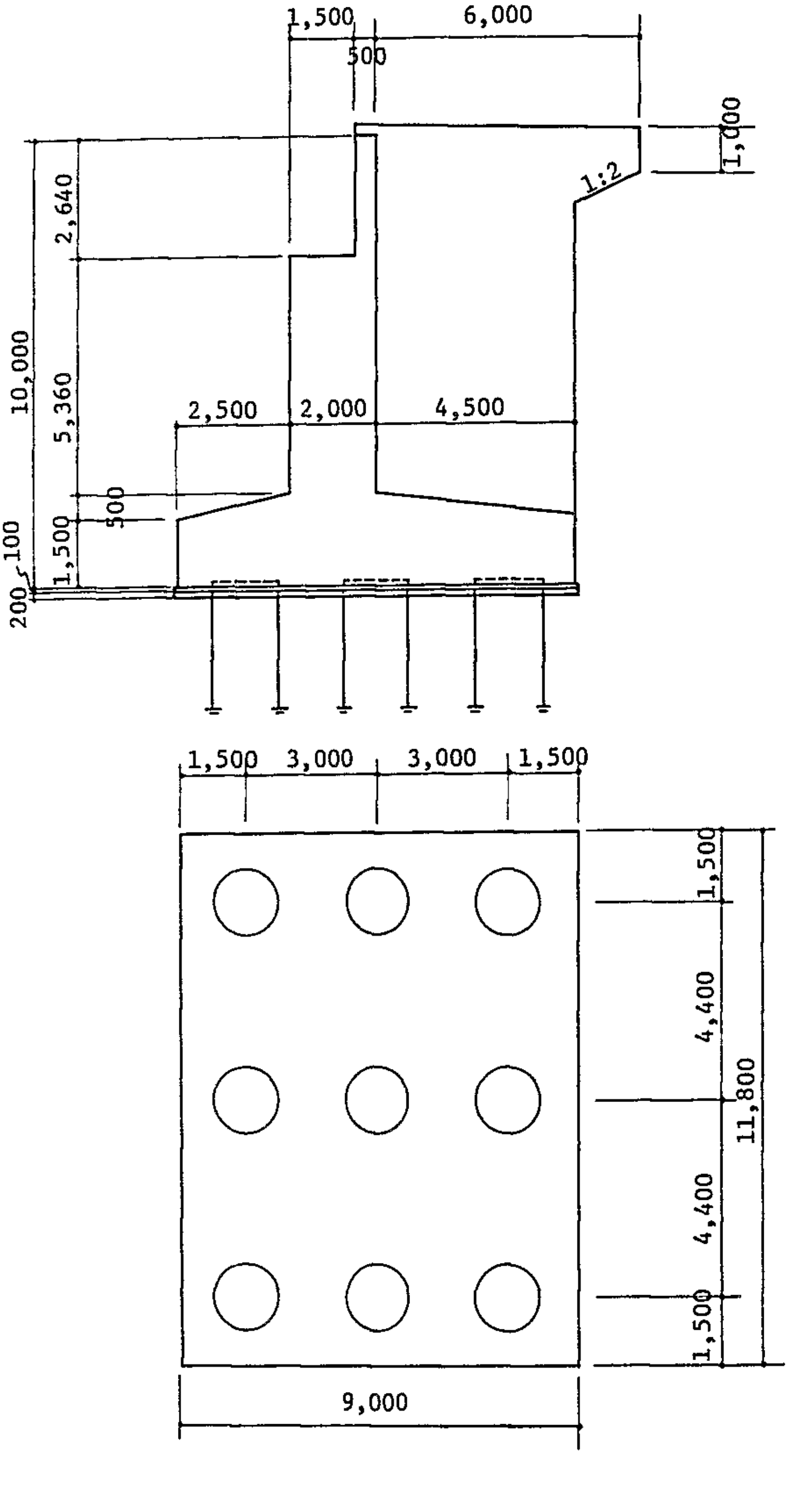
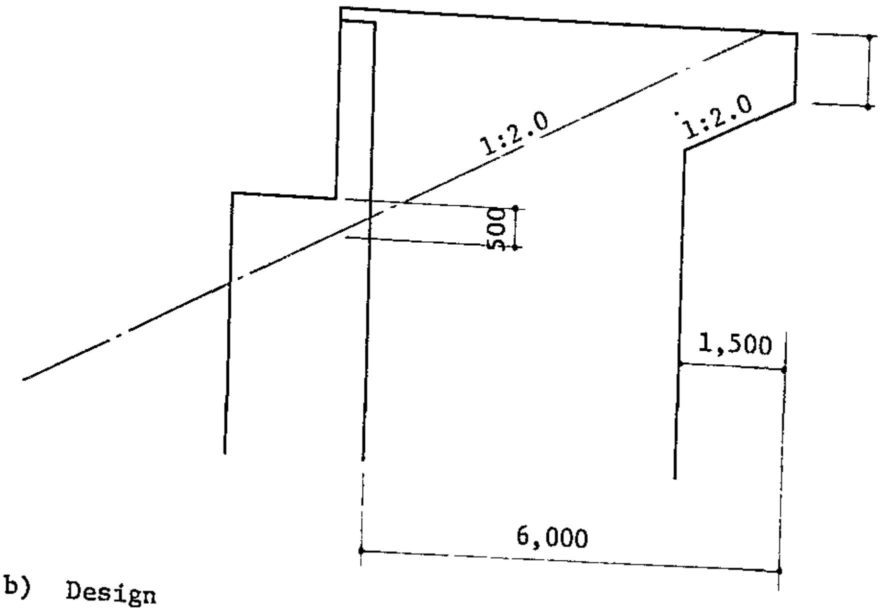


Fig. 7-1

Shape of the Wing Wall

Since earthwork is yet to be planned, the stable slope gradient of embankment is determined on the basis of assumption as follows:



b) Design

Reinforcement

Fig. 7-2

Reinforcement will be spaced basically in 125cm pitch. Section of the abutment is established adopting D25 bar as the main reinforcing material.

Stress in Structural Members

(Figures in the lower columns are allowable values)

		N (t)	M (tm)	S (t)	σc (kg/cm^2)	σs (kg/cm^2)	τ (kg/cm^2)	As (cm^2)	Remarks
Parapet		-	13.10	6.99	54.8	1597	2.0	DA0425	Normally
					70	1600	8.5	22.92	a = 1.00
Wall		-	120.43	36.52	35.1	2138	21	D25ctc250	Under seisnic load
					105.0	?700	12.75	D19 " 31.73	a = 1.50
Footing	Front toe	-	111.02	71.75	31.1	1620	4.2	D25ctc125	Normal case+temperature change
					80.5	1840	9.78	40.54	= 1.15
	Rear toe	-	130.76	12.29	38.0	2317	0.7	D25ctc250	under seismic load
					105.0	2700	12.25	D19 " 31.73	a = 1.50
Wing wall	X direction		23,01	-	65.3	1443		D25ctc125	normally
					70.0	1600	-	40.54	s = 1.00
	Y direction	-	22.74	-	64.6	1426	-	D25ctc125	"
					70.0	1600	-	40.54	a = 1.00
Cast-in-sita pile		78.12	202.29	70.30	100.37	2654	5.19	D29 n=25	under seismic load
					120.0	2700	13.5	160.60	with min N value

Reaction of Piles

		A1	A2
Vertical reaction	Max (t)	389.3	389.3
		470.4	705.7
	Min (t)	78.1	78.1
		-330.8	-330.8
Horizontal displacement	(m)	0.015	0.015
		0.025	0.025

3. Design of Piers

a) Stress of Structural Member

(1) Pier P_1

		N (t)	M (t.m)	S (t)	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)	As (cm²)	Remarks
Beam			343.08	711.03	36.1	1,462	17.8	D29 n=23	under normal condition
					70	1,800	6.5	147.75	vertically
Column		1,475.39	1,285.43	147.75	53.1	738	0.3	@250 258 D19 n=48	under normal condition in axial direction
					70	1,800	6.5	137.52	
Foot-ings	Axially		149.95	120.87	28.6	1,686	5.5	D25@125	under normal condition
					80.5	1,840	9.8	40.54	
	Laterally		74.07	103.45	22.1	2,291	4.6	D12@200	under seismic load
					105	2,700	12.8	14.33	
Cast-in-site		79.42	49.48	33.65	25.5	378	1.8	@188.5 D25 n=20	under seismic load
					120	2,700	10.5	101.34	with min N value

(2) Pier P2

		N (t)	M (t.m)	H(s) (t)	σc (kg/cm^2)	σs (kg/cm^2)	τ (kg/cm^2)	As (cm^2)	Remarks
Column	head				111.32	2642.5	0.0	D32 η =46 As-1=365.332 D32 η =19	under construction normal condition
		2750.0	5160.00	0.00	150.0	2700	15.0	As-1=150.898 516.23	
					140.6	2901.7	1.5	D32 η =92 As-1=730.664 D32 η =38	under construction under seismic load
	foot	3370.00	8450.00	205.00	165.0	2970	16.5	As=2=301.796 1032.46	
Caisson					64.9	2413	9.9	D32 @125	secondary state
	top slab		934.18	299.30	120.0	2700	13.5	127.00	under seismic load
	caisson vertucally				11.2	45.3		D19 η =144	after seismic completion
		5882.67	16393.75		90.0	2700		412.56	
					16	142		D16@250	under seismic load
		174.83	59.81		90.0	2700	9	7.94	
					34.0	1647	1.8	D25 η =9	under construction
			132.00	32.40	90.0	2700	9	45.603	in vertical direction
					56.3	255.8		D16@250	under construction
		115.48	19.54		90.0	2700	9	7.94	
					21.6	81.2		D16@250	under construction
		155.81	25.34		90.0	2700	9	7.94	

(3) Pier P3

		N (t)	M (t.m)	H(s) (t)	σc (kg/cm²)	σs (kg/cm²)	τ (kg/cm²)	As (cm²)	Remarks
Column	head				111.32	2642.5	0.0	D32 n=46 As-1=365.332 D32 n=19	under construction normal condition
		2750.00	5160.00	0.00	150.0	2700	15.0	As-2=150.898 516.23	
	foot				140.6	2901.7	1.5	D32 n=92 As-1=730.664 D32 n=38 As-2=301.796	under construction under seismic load
		3370.00	8450.00	205.00	165.0	2970	16.5	1032.46	
Caisson	top slab				64.9	2413	9.9	D32@125	Secondary state
			934.18	299.30	120.0	2700	13.5	127.08	under seimic load
	caisson vertically				13.3	73.5		D19 n=144	after completion under seismic load
		5882.67	18812.32		90.0	2700		412.56	
					25	226		D16@250	under seismic load
		325.10	35.78		90.0	2700	9.0	7.94	
					45.1	2183	4.9	D25 n=9	under construction
	.		175.20	86.40	90.0	2700	9.0	45.603	in vertical direction
					56.3	255.8		D16@250	under construction
		115.48	19.54		90.0	2700	9.0	7.94	
	parapet				21.6	81.2		D16@250	under construction
		155.81	25.34		90.0	2700	9.0	7.94	

(4) Pier P4

		N (t)	M (t.m)	S (t)	σc (kg/cm^2)	σs (kg/cm^2)	τ (kg/cm^2)	As (cm^2)	Remarks
Beam			343.08	711.03	36.1	1.462	17.8	D29 n=23	under normal condition
					70	1.800	6.5	147.75	vertically
Column		1,447.49	1,137.68	147.75	44.2	426	0.2	D19 @350	under normal condition
					70	1.800	6.5	258 n=48	in axial direction
Foot-ings	axially		144.96	116.88	27.5	1.624	5.3	D25@125	under normal condition
					80.5	1.840	9.8	40.54	
	laterally		70.84	99.25	21.1	2.189	4.4	D19@200	under seismic load
					105	2.700	12.8	14.33	
Cast-in-sita		85.25	54.89	33.09	28.6	441	1.8	@188.5 D25 n=20	under seismic load
					120	2.700	10.5	101.34	with min N value

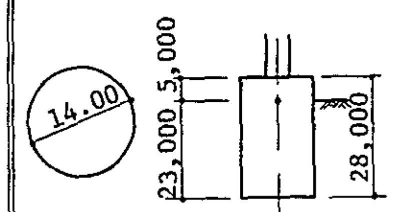
b) Reaction of Piles

(Figures in lower columns show allowable value)

			Pier P_1	Pier P_4
Cast-in-situ piled foundation	Vertical reaction	Max. (t)	425.4 478.0 (717.1)	410.2 507.3 (761.0)
		Min. (t)	79.4 -230.2	85.3 -276.2
	Horizontal displacement	(m)	0.009 0.025	0.009 0.025

c) <u>Reaction of Caissons</u>

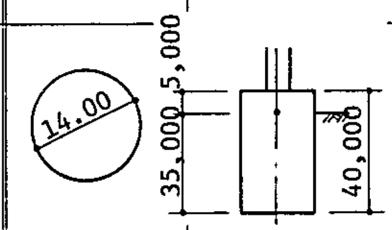
(1) Caisson P_2

Input data					Normally		Under earthquake
Process order	Item	Code used in porgram	Symbol used in clause 4.3	Unit	After completion	Under construction	After completion
1	Data number	NO.					
2	Shape of caisson base for rectangular base: 1 for circular base: 2 for oval base of case (i) in Fig. (14): 3 for oval base of case (ii)(iii) in Fig. (14) are input. The shift from case (ii) of oval base to case (iii) will be implemented within the program.	KATA					
3	Vertical force in the hypotheticle ground surface, acting against the caisson	N	N	t	6,292.67	5,252.67	5,882.67
4	Horizontal force in the hypotheticle ground surface, acting against caisson	H	H	t	0	0	705.93
5	Moment in the hypotheticle ground surface, acting against the caisson	M	M	t/m	3,073.00	6,171.00	12,106.02
6	Weight of caisson per unit length	W	w	t/m	332.53	332.53	332.53
7	Buoyancy acting on the caisson	U	U	t	4,310.26	4,310.26	4,310.26
8	Lateral width	A2	2a	m	14.00	14.00	14.00
9	Frontage width	B2	2b	m	14.00	14.00	14.00
10	Thickness of first layer of circumferential soil	L1	L_1	m	7.00	7.00	7.00
11	" 2nd "	L2	l_2	m	16.00	16.00	16.00
12	" 3rd "	L3	l_3	m	0	0	0
13	Horizontal ground reaction coefficient of 1st layer of frontal soil	KH1	K_{H1}	t/m^3	406	406	812
14	" " 2nd "	KH2	K_{H2}	t/m^3	1,192	1,192	2,384
15	" " 3rd "	KH3	K_{H3}	t/m^3	0	0	0
16	Vertical ground reaction coefficient of caisson sub-base soil	KV	Kv	t/m^3	2,286	2,286	4,572
17	Shear resilent coefficient of caisson sub-base soil	KS	Ks	t/m^3	653	653	1,306
18	Horizontal seismic coefficient	K	k		0	0	0

Output data					Normally		Under earthquake
Process order	Item	Code used in program	Symbol used in clause 4.3	Unit	After completion	Under construction	After completion
1	Frontal ground reaction stress	P12	P_{12}	t/m^2	0.67	1.35	5.94
		P21	P_{21}		1.97	3.96	17.46
		P22	P_{22}		-1.33	-2.68	8.17
		P31	P_{31}		0	0	0
		P32	P_{32}		0	0	0
2	Sub-base ground reaction stress in the case of triangular distribution only Q1 will be output.	Q1	q_1	t/m^2	65.34	61.38	81.40
		Q2	q_2		59.78	50.22	38.38
3	Width of caisson base reaction not entailing the floating of the caisson	D	d	m	—	—	—
4	Value of angle in Fig. 14	BA	β		—	—	—
5	Shear of caisson base	R	R	t	-112.69	-226.30	-689.11
6	Displacement of caisson crown	DE1	σ_1	m	0.0028	0.0057	0.012
7	" " " base	DE2	σ_2	m	-0.0011	-0.0022	-0.0034
8	Angle of rotation of caisson	T	θ	degree	0.01	0.02	0.0385
9	Depth of rotation center of base	E	h	m	16.545	16.545	17.901
10	Frontal ground reaction stress within 0 y l_1 will be output in 1/10 l inervals	Y	y	m	—	—	—
		PY	P_y	t/m	—	—	—

Note: Items 3.4 will be input only in the case of triangular distribution.

(2) Caisson P_3

Input data					Normally		Under earthquake
Process order	Item	Code used in program	Symbol used in clause 4.3	Unit	After completion	Under construction	After completion
1	Data number	NO.					
2	Shape of caisson base for rectangular base: 1 for circular base: 2 for oval base of case (i) in Fig. (14): 3 for oval base of case (ii)(iii) in Fig. (14) are input. The shift from case (ii) of oval base to case (iii) will be implemented within the program.	KATA					
3	Vertical force in the hypotheticle ground surface, acting against the caisson	N	N	t	6,292.67	5,252.67	5,882.67
4	Horizontal force in the hypotheticle ground surface, acting against the caisson	H	H	t	0	0	705.93
5	Moment in the hypotheticle ground surface, acting against the caisson	M	M	t/m	3,073.00	6,171.00	12,106.02
6	Weight of caisson per unit length	W	w	t/m	323.29	323.29	323.29
7	Buoyancy acting on the caisson	U	U	t	6,157.52	6,157.52	6,157.52
8	Lateral width	A2	2a	m	14.00	14.00	14.00
9	Frontage width	B2	2b	m	14.00	14.00	14.00
10	Thickness of 1st layer of circumferential soil	L1	l_1	m	15.50	15.50	15.50
11	" 2nd "	L2	l_2	m	10.00	10.00	10.00
12	" 3rd "	L3	l_3	m	9.50	9.50	9.50
13	Horizontal ground reaction coefficient of 1st layer of frontal soil	KH1	K_{H1}	t/m^2	301	301	601
14	" " 2nd "	KH2	K_{H2}	t/m^2	568	568	1,137
15	" " 3rd "	KH3	K_{H3}	t/m^2	1,448	1,448	2,896
16	Vertical ground reaction coefficient of caisson sub-base soil	KV	Kv	t/m^2	2,286	2,286	4,572
17	Shear resilent coefficient of caisson sub-base soil	KS	Ks	t/m^2	653	653	1,306
18	Horizontal seismic coefficient	K	k		0	0	0

Output data					Normally		Under earthquake
Process order	Item	Code used in program	Symbol used in clause 4.3	Unit	After completion	Under construction	After completion
1	Frontal ground reaction stress	P12	P_{12}	t/m^2	0.41	0.84	4.85
		P21	P_{21}		0.78	1.58	9.17
		P22	P_{22}		0.15	0.30	2.58
		P31	P_{31}		0.39	0.78	6.57
		P32	P_{32}		-1.15	-2.31	-9.39
2	Sub-base ground reaction stress, in the case of triangular distriubtion only Q1 will be output.	Q1	q1	t/m^2	76.17	71.22	90.29
		Q2	q2		72.58	64.02	53.14
3	Width of caisson base reaction not entailing the floating of the caisson	D	d	m	—	—	—
4	Value of angle in Fig. 14	BA	β		—	—	—
5	Shear of caisson base	R	R	t	-79.87	-160.39	-651.93
6	Displacement of caisson crown	DE1	σ_1	m	0.0031	0.0062	0.017
7	" " " base	DE2	σ_2	m	-0.0007	-0.015	-0.0032
8	Angle of rotation of caisson	T	θ		0.0064	0.0129	0.0332
9	Depth of rotation center of base	E	h	m	27.91	27.91	29.412
10	Frontal ground reaction stress within 0 y l_1 will be output in 1/10 $_1$ intervals	Y	y	m	—	—	—
		PY	Py	t/m^2	—	—	—

Note: Items 3.4 will be input only in the case of triangular distribution.

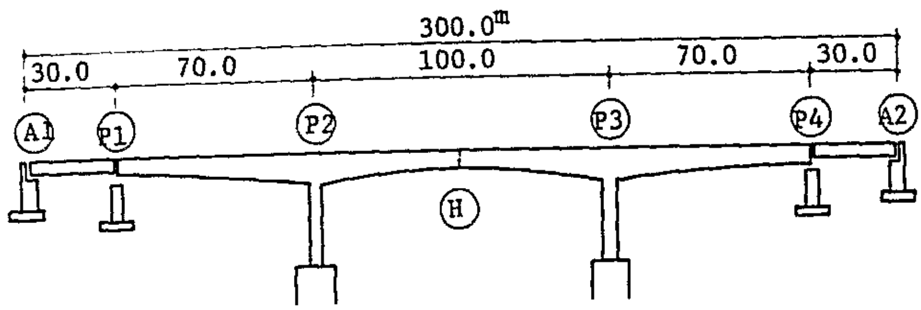
Chapter 8. DESIGN OF APPURTENANCES

1. Bearing Devices

High strength copper-alloy bearing plate support which is most commonly used currently for P.C. bridges of a similar scale to this one.

The hinges to be installed at the center of P.C. box girder span will be designed as bearing plate support which conveys only shearing force.

(1) Design Reaction



		Simple supported composite beam		Continuous box girder	
		A1 · A2	P1 · P2	P1 · P2	H
Number of bearings		4	4	2	2
Dead load reaction		92.6	92.6	305.6	——
Live load reaction	Max.	29.4	29.4	76.4	* 35.2
	Min.	——	——	-28.9	*-35.2
Total reaction	Max.	122.0	122.0	382.0	* 35.2
	Min.	——	——	276.7	*-35.2

* denotes shearing force

(2) A_1 A_2 P_1 P_4 Simply Supported P.C. Beam

Movable and Fixed Bearings, R = 122 ton

Reaction

Total reaction	R	$= 122.0$ ton
Dead load reaction	Rd	$= 92.6$ ton
Live load reaction	$R(\ell + i)$	$= 29.4$ ton

Axial Horizontal Force (In motion) ... frictional force on sliding surface (for movable bearing only)

$R_{H1f} = f \cdot R = 0.15 \times 122.0t = 18.3$ ton

Axial Horizontal Force (under seismic load)

Fixed bearing $R_{H1e} = K_H \cdot Rd = 0.2 \times (2 \times 92.6) = 37.1$ ton

Movable bearing $R_{H1e} = K_H \cdot Rd = 0.2 \times 92.6t = 18.6$ ton

Lateral Horizontal Force (under seismic load)

$R_{H2e} = K_H \cdot Rd = 0.2 \times 92.6t = 18.6$ ton

Lift (under seismic load) ... assume at 10% of dead load

$V = 0.1 \cdot Rd = 0.1 \times 92.6t = 9.3$ ton

Displacement (for movable bearing only)

Computed amount of displacement

$e_1 = 30mm$

Design dispalcement

$e_2 = e_1 + 20 = 30 + 20 = 50mm$

Possible amount of displacement

$e = e_2 + 40 = 50 + 40 = 90mm$

Design Horizontal Seismic Coefficient $K_H = 0.2$

Design Frictional Coefficient $f = 0.15$

Allowable Bearing Stress for Concrete

Upper member ba = 0.3 ck = $0.3 \times 350 = 105$ kg/cm^2

Lower member ba = 0.3 ck = $0.3 \times 210 = 63$ kg/cm^2

σck: Design standard strength of concrete

(3) P_1 P_4 Main bridge

Movable Bearings, R = 382 ton

Reaction

Total reaction	R	= 382.0 ton
Dead load reaction	Rd	= 305.6 ton
Live load reaction	R (ℓ + i) =	76.4 ton

Axial Horizontal Force (In motion) ... frictional force on sliding surface (for movable bearing only)

$R_{H1f} = f \cdot R = 0.15 \times 386\ t = 57.3$ ton

Axial Horizontal Force (under seismic load)

$R_{H1e} = K_H \cdot (Rd + \text{Additional load})$

$= 0.2 \times (305.6t + 12.6t) = 63.7$ ton

Lateral Horizontal Force (under seismic load)

$R_{H2e} = K_H \cdot Rd = 0.24 \times 305.6t = 61.1$ ton

Lift (under seismic load) ... assume at 10% of dead load

$V = 0.1 \cdot Rd = 0.1 \times 305.6t = 30.6$ ton

Displacement (for movable bearing only)

Computed amount of displacement

$e_1 = 90$ mm

Design displacement

$e_2 = e_1 + 20 = 90 + 20 = 110$ mm

Possible amount of displacement

$e = e_2 + 40 = 110 + 40 = 150$ mm

Design Horizontal Seismic Coefficient

$K_H = 0.2$

Design Frictional Coefficient

$f = 0.15$

Allowable Bearing Stress for Concrete

Upper member $\quad ba = 0.3 \quad ck = 0.3 \times 350 = 105$ kg/cm^2

Lower member $\quad ba = 0.3 \quad ck = 0.3 \times 210 = 63$ kg/cm^2

(4) Horizontally sliding Hinge at H

Vertical shearing force: S

$$S = \frac{70.4^t}{2} = 35.2^t = 36^t$$

Displacement

Assuming the temperature at the time of installation to be equall to the standard temperature

Amount of expansion due to temperature change: $\Delta \ell t$

$\Delta \ell t = \pm 10.0 \times 2 = \pm 20.0$ mm

Drying shrinkage: s

$\Delta \ell s = -6.0 \times 2 = -12.0$ mm

Contraction due to creep: c

$\Delta \ell c = =11.1 \times 2 = -22.2$ mm

Contraction due to elastic deformation: p

$\Delta \ell p = -9.2 \times 2 = -18.4$ mm

Total

Expansion $\Delta\ell$ = +20 mm

Contraction $\Delta\ell$ = -72.6 mm

In the design, the following values which are allowed with marging of safety will be adopted

Expansion $\Delta\ell$ = 20 + 10 + = +30 mm

Contraction $\Delta\ell$ = 72.6 + 30 = -105 mm

Where + indicates amount of change of central hinge in protruding direction and - indicates that in retracting direction.

Design standard strength of concrete: σck

$\sigma ck = 350\ kg/cm^2$

P.C. Steel Bar Utilized and Amount of Prestressing force (P)

No.2 (95/120) for ϕ32 use

Prestressing force

P = 40 t/piece x 6 pieces = 240 t

2. Expansion Joints

(1) Selection of Type of Expansion Joints

Table Classification of Expansion Joints

	Type	Classification	Remarks
Butt joint type	Dummy joint	- Dummy joint	Displacement absobed by deformation of asphalt pavement
	Butt joint pre-installed type	- Filler joint - Angle reinforced joint - Steel stiffened joint	Butt joint device installed prior to execution of pavement
	Butt joint post-installed type	- Cut-off joint - Coupling joint - Hama-Highway type joint - Others	Butt joint device installed after execution of pavement
Bearing type	Rubber joint type	- Hama-Highway Joint - Cold and Bloft joint - Neosumi joint - Others	Constitute of rubber and steel elements, support wheel load at the opening between adjacent slabs
	Steel type	- Steel finger joint - Steel lap joint	Using face plate or finger plate made of steel
	Special type	- Denmark type etc.	

Steel finger joint is chosen for use in this project for its durability and ease of maintenance, provided properly manufactured and installed.

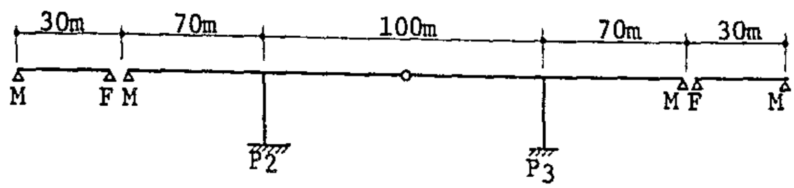
(2) Design Criteria

Rear wheel load P = 8,000kg (as stipulated in "Highway Bridge Specification" item 1.8.4, Live load T-20)

Impact coefficient i = 1.0 (from "Standard Design of Expansion Joints" of Japan Highway Corporation Table 1-1)

Temperature change   Yearly average ± 20°C

Length of beam used for calculation of displacement



Amount of expansion (from "Standard Design of Expansion Joints" of Japan Highway Corporation, Table 4-1)

° Elastic deformation due to prestressing develops befor installation, and is therefore ignored.

° In consideration of contraction due to creep and drying shrinkage, the concrete age should be 2 - 3 months. When expansion joint is placed. The decrement modulus is taken at = 0.5.

° Amount of expansion (taking the temperature of the time of installation to be equal to the yearly average temperature)

a) Amount of expansion due to temperature change ($\Delta \ell t$)

Expansion : $\Delta\ell_t = 0.2 \times \ell$ mm

Contraction: $\Delta\ell t = 0.2 \times \ell$ mm

b) Amount of expansion due to drying shrinkage ($\Delta \ell s$)

Contraction: $\Delta\ell_s = 0.1 \times \ell$ mm

c) Amount of expansion due to creep ($\Delta \ell c$)

Contraction: $\Delta\ell_c = 0.2 \times \ell$ mm

d) Allowance of safety margin; 20% of the sum of the above items

e) Allowance to absorb rotating displacement of girder end due to load of pavement and live load, execution error etc.: ± 5mm

			At center H	$P_1 \cdot P_4$	$A_1 \cdot A_2$
Length of beam			100m	70m	30m
Due to temperature change		Expansion	+20mm	+14mm	6mm
		Contraction	-20	-14	6
Due to drying shrinkage		Contraction	-10	- 7	- 3
Due to creep of concrete		Contraction	-20	-14	- 6
Safety margin allowance	For temperature change	Expansion	+ 4	+ 2.8	- 1.2
		Contraction	- 4	- 2.8	- 1.2
	For drying shrinkage	Contraction	- 2	- 1.4	- 0.6
	For creep of concrete	Contraction	- 4	- 2.8	- 1.2
Additional allowance			± 5	± 5	± 5
Total ($\Delta\ell$)		Expansion	+29mm	+21.8mm	+12.2mm
		Contraction	-65mm	-47.0mm	-23.0mm

(3) Allowable Stress (From "Standard Design of Expansion Joints of Japan Highway Corporation)

1. Steel Material SM41B --- JIS G 3106

SS41 --- JIS G 3101

NSD41 --- Deformed steel bat. Chemical ingradients and mechanical properties to comply with requirements of "Arc stud dowel welding manual (draft)" of Japan Highway Corporation.

Allowable tensile stress	σta = 1,400 kg/cm^2
Allowable compressive stress	σca = 1,400 "
Allowable shearing stress	τa = 800 "
Allowable shearing stress for fillet welding	τa = 800 "

2. Slab and Abutment Concrete (Design Standard Strength $\sigma ck = 240\ kg/cm^2$)

Allowable tensile stress	$\sigma ba = 0.3 \times \sigma ck = 120 kg/cm^2$
Allowable shearing stress	$\tau a = 3.9\ kg/cm^2$
Allowable bond stress	
Anchor plate (F.B.)	$\sigma aa = 8\ kg/cm^2$
Anchor bar (Stud)	$\sigma aa = 16\ kg/cm^2$

Chapter 9. MAIN MATERIALS

1. List of Material Quantities for Superstructure (Main bridge)

Material name	Type	Unit	Quantity	Remarks
Concrete	$\sigma ck=350kg/cm^2$	m^3	2,592.0	Main beam
		"	345.4	Surfacing, railing, sidewalk
	Total	m^2	2,937.4	
Formworks		"	7,737.5	Main beam (end shuttering)
		"	1,228.8	Surface, railing, sidewlak
		"	8,966.3	
Reinforcement	D19	kg	31,860	Main beam, surfacing, railing, etc.
	D16	"	11,826	
	D13	"	180,348	
	D10	"	9,528	$(90.1\ kg/m^3)$
	Total	"	233,562	
P.C. steel baton (SBPC95/120)	Main baton	t	146.557	SBPC95/120 ϕ33 mm
	Diagonal baton	"	20.828	
	Lateral tie baton	"	32.944	$(77.3\ kg/m^3)$
	Total	"	200.329	
Sheath	ϕ39mm	m	31,755.9	
Coupler sheath = 250	ϕ70mm	piece	1,944	
= 350	"	"	788	
= 450	"	"	88	
with nozzle = 250	"	"	616	
" = 350	"	"	120	
" = 450	"	"	——	
Total		"	3,024	
Coupler	ϕ60mm L=110mm	piece	3,024	S35C
Anchor plate	Tensed side	"	1,734	Including G anchor
	Fixed side	"	1,562	
Grout	ϕ39mm	m	31,755.9	

Material name		Type	Unit	Quantity	Remarks
Pavement		Average t=90mm	m^2	1,920.0	For carriageway
Drainage facility	Gully		kg	2,976.0	48 pieces
	Drainage pipe				
Expansion joint		Finger	kg	12,495.8	P_1, P_4 Central hinges
Bearings etc.		382 ton shoe	"	6,069.6	P1, P4 (N=4 pieces)
		Central hinge	"	2,708.8	(N=2 sets)
		Horizontal shoe	"	112.0	(N=2 ")
Mounting pieces for appurtenants		Shape steel t=6mm B=50mm	kg		
		Bolt	piece		

2. <u>List of Material Quantity for Superstructure</u> (Symply supported PC Composite Beam)

		Material name	Type	Unit	Average per bridge	Total	Remarks
Main beam		Concrete	kg/cm^2 σck-350	m^3	124.0	248.0	
				m^2	667.6	1,335.2	
		Reinforcement (SD30)	D10	kg	2,719.2	5,438.4	
			D13	"	3,268.8	6,537.6	
			D16	"	1,173.2	2,346.4	
			Total	"	7,161.2	14,322.4	
	P.C. steel material	P.C. steel wire	12-ϕ7	kg	5,608.0	11,216.0	
		Sheath	ϕ45	m	1,534.8	3,069.6	
		Grout	ϕ45	"	1,534.8	3,069.6	
		Anchoring piece	For 12-ϕ7 use	set	104	208	
Cross beam		Concrete	kg/cm^2 ck=240	m^3	23.3	46.6	
		Formwork		m^2	142.3	284.6	
		Reinforcement (SD30)	D10	kg	——		
			D13	"	1,148.5	2,297.0	
			D16	"	——		
			Total	"	1,148.5	2,297.0	
	P.C. steel material	P.C. steel wire	12-ϕ7	kg	360.6	721.2	
		Sheath	ϕ45	m	97.1	194.2	
		Grout	ϕ45	"	97.1	194.2	
		Anchoring piece	For 12-ϕ7 use	set	20	40	
Slab		Concrete	kg/cm^2 ck=240	m^3	76.4	152.8	
				"	——		
			Total	"	76.4	152.8	
		Formwork		m^2	243.7	487.4	
				"	——		
			Total	"	243.7	487.4	

Material name			Type	Unit	Average per bridge	Total	Remarks
Slab	Reinforcement (SD30)		D10	kg	——	——	
			D13	"	4,192.3	8,384.6	
			D16	"	8,756.4	17,512.8	
			Total	"	12,948.7	25,897.4	
	P.C. steel material	P.C. steel wire	12-ϕ7	kg	——	——	
		Sheath	ϕ45	m	——	——	
		Grout	ϕ45	"	——	——	
		Anchoring piece	For 2-ϕ7 use	set	——	——	
Surfacing, Sidewalk, Railing etc.	Concerete		σck=240 kg/cm^2	m^3	43.0	86.0	
	Formwork			m^2	158.1	316.2	
	Reinforcement (SD30)		D10	kg	1,106.5	2,213.0	
			D13	"	1,533.6	3,067.2	
			Total	"	2,640.1	5,280.2	
Pavement			Asphalt	m^2	238.8	477.6	t=75 mm
Drainage facility	Gully			kg	372.0	744.0	12 pieces
	Drainage pipe						
Expansion joint			Finger	kg	3,754.8	7,509.6	A_1, A_2
Bearings			Mov	kg	1,304.4	2,608.8	4 main
			Fix	"	1,256.0	2,512.0	

3. List of Material Quantity for Substructure

Material		Specification	Unit	A_1	P_1	P_2	P_3	P_4	A_2	Total	Remarks
Concrete	Body	$\sigma ck=300kg/cm^2$	m^3	—	—	281.25	281.25	—	—	562.50	
		$\sigma ck=210$ "	"	188.80	123.78	—	—	112.62	188.80	614.00	
	Footing	$\sigma ck=210$ "	"	192.34	167.02	—	—	167.02	192.34	718.72	
	Caisson	$\sigma ck=240$ "	"	—	—	395.64	395.64	—	—	791.28	
		$\sigma ck=180$ "	"	—	—	2,827.83	4,009.47	—	—	6,837.30	
	Total		"	381.14	290.80	3,504.72	4,686.36	279.64	381.14	9,523.80	
	Founda-tion	ck=100 "	"	9.45	6.48	—	—	6.48	9.45	31.86	
Form-works	Body	Straight	m^2	373.29	92.01	291.06	291.06	87.01	373.29	1,507.72	
		Curve	"	—	64.82	—	—	56.97	—	121.79	
	Footing		"	68.00	73.35	—	—	73.35	68.00	282.70	
	Caisson	Curve	"	—	—	1,958.96	2,802.98	—	—	4,761.94	
		Built-in formwork	"						—	110.78	
	Total		"	441.29	230.18	2,305.41	3,149.43	217.33	441.29	6,784.93	
	Founda-tion con-crete		"	4.24	3.48	—	—	3.48	4.24	15.44	
Rein-foce-ment	D32		kg	—	—	90,128	90,128	—	—	180,256	
	D29		"	—	1,568	—	—	1,568	—	3,136	
	D25		"	12,275	3,407	20,851	20,851	3,407	12,275	73,066	
	D19		"	2,260	2,300	19,110	28,009	2,191	2,260	56,130	
	D16		"	5,736	3,810	23,429	29,300	3,727	5,736	71,738	
	D13		"	1,308	789	5,823	8,311	774	1,308	18,313	
	D10		"	44	—	—			44	88	
	Total		"	21,623	11,874	159,341	176,599	11,667	21,623	402,727	
Cobble stone			m^3	18.90	12.97	—	—	12.97	18.90	60.74	
Scaffolding			m^3	278.40	304.30	449.43	449.43	268.50	278.40	2,028.46	
Timbering			"	—	107.95	—	—	90.95	—	198.90	
Mortaring			"	0.036	0.108	—	—	0.108	0.036	0.936	

Material			Specification	Unit	A1	P1	P2	P3	P4	A2	Total	Remarks
Earth work	Excavation			m^3	126.0	315.0			275.0	126.0	842.0	
	Surplus soil			"	111.2	193.1			188.1	111.2	603.6	
	Back-fill soil			"	14.8	121.9			86.9	14.8	238.4	
Cast-in-situ piling	Design length		1,500 ϕ	m	45.0	30.0	——	——	42.0	45.0	——	
	Concrete volume		$\sigma ck=300kg/cm^2$	m^3	715.69	318.09			445.32	715.69	2,194.79	
	Reinforce-ment	D29	SD-30	kg	11,340					11,340	22,680	
		D25	"	"	28,800	16,200			22,404	28,800	96,204	
		D19	"	"	1,728	786			1,092	1,728	5,334	
		D16	"	"	6,786	3,486			4,260	6,786	21,318	
		Total	"	"	48,654	20,472			27,756	48,654	145,536	
	Spacer		FB.PL 50x6x370	"	774	342			468	774	2,358	
	Piling excava-tion			m^3	709.3	313.9			441.1	709.3	2,173.6	
Cut-off wall	Concrete			"			315.57	315.57			631.14	
	Form-work			m^2			1,051.90	1,051.90			2,103.80	
	Reinforce-ment	D16	SD-30	kg			13,730	13,730			27,460	
		D13		"			618	618			1,236	
		Total		"			14,348	14,348			28,696	
Cutting edge	Flange		SS41	kg			12,359	12,359			24,718	
	Rein-force ment D19		SD-30	"			486	486			972	
	Bolt nut			"			99	99			198	