Tertiary	Weathered Sedimentary Rock	Sw
"	Sandstone & Shale	Ns & Sh

The properties of each layer are summarized in Table 6.3.2.

6.3.5 Planimetric Surveying

Planimetric surveying was carried to make topographical maps of scale 1/500 and scale 1/1,000 for the detailed bridge location study and the route location study for road and railway. The topographical maps were developed with one meter contour line.

6.3.6 Results of Laboratory Test

The laboratory tests on the samples obtained from:

1) the split sampler for SPT
2) the single-tube (ø2") for coring
3) the thin walled tube (ø3") for open drive sampling
4) the test pits for subgrade materials (Route 1, east bank)

have been carried out at the Research and Soil Laboratories (CC), Rangoon.

All tests followed the method specified in the ASTM or equivalent.

The quantities of laboratory tests and the engineering properties of soils are summarized in Tables 6.3.3 and Figs. 6.3.5 - 6.3.9, respectively, and the individual test results are shown in the Appendix Tables 6.3.4 - 6.3.19.

<u>Gradation</u>

The typical grain-size distribution curves of Sd_1, Sd_2 and Sd_3 layer are shown in the Fig. 6.3.10. Most of the sandy soil in the riverbed is classified as SP (poorly graded sand and gravelly sand, little or no fine sand).

Table 6.3.2 PROPERTIES OF LAYER

	Geological Age	Type of Soil	Symbol	Color	N-value (Frequent Range)	Relative Density or Consistency	General Description
Quaternary	Recent Alluvium	Sandy Soil (Silty Sand) ~ (Sand & Gravel)	Sd_1	Brown ~ Dark Gray	2 - 15 (4 - 10)	Very Loose Loose	Layer thickness: 12.2 m - 9.4 m (Route 1), 12.2 m -7.9 m (Route 2) Sand: fine to coarse grained (mainly fine) Gravel: Sub-rounded quartz, silified wood, sandstone ϕ 2 - 10 m/m, max. ϕ 15 m/m Mixed with a little mica, thin layer of clay and silt.
Quaternary	Older Alluvium	Sandy Soil (Silty Sand) ~ (Sand & Gravel)	Sd_2	(Yellowish) Brown ~ Dark Gray	13 - 46 (13 - 28)	Medium Dense	Layer thickness: 0 m - 24 m (Route 1), 0 m - 23.2 m (Route 2) Sand: fine to coarse grained (mainly fine to med.). Gravel: Sub-rounded quartz, sandstone, finestone ϕ 2 - 10 m/m Mixed with a little mica and decayed wood. Exist lense of very stiff clayish silt with 1.5 m - 3.2 m thickness at Route 1.
Quaternary	Older Alluvium	Sandy Soil (Silty Sand) ~ (Sand & Gravel)	Sd_3	Gray ~ Dark Gray	23 - 750 (≥ 34)	Dense Very Dense	Layer thickness: 0 m - 22.7 m (Route 1), 0 - > 19.8 m (Route 2) Sand: fine to coarse grained (mainly med. to fine) Gravel: sub-rounded quartz, sandstone, finestone. ϕ 2 - 10 m/m (max. ϕ26 m/m at Route 2) Mixed with a little mica and decayed wood. Exist lense of hard clayish silt with 1.5 m - 3.0 m thickness at Route 1. The layer with N = 30 - 50 is observed about 6 m thick at Route 1 and 9.2 m - 3.5 m thick at Route 2.
Tertiary	Miocene	Weathered Sedimentary Rock (Clay - Silty Sand)	Sw	Brown ~ Yellowish Brown	4 - 39	Soft - Hard & Med.-Dense	Soil-like, heavily weathered sedimentary rock (= sedantary soil). Exist at the east bank of Route 1. Over lower portion, cohesive soil predominates. Over hilly area, silty sand with gravels (ϕ 2 - 10 cm max. ϕ 15 cm, quartz silified wood, sub-rounded predominates. Sand is fine to medium grained.
Tertiary	Miocene	Sandstone & Shale	Ss & Sh	Yellowish Brown Gray (Sandstone) Brownish/ Blueish Gray ~ Gray (Shale)			Sandstone: Laminated in the shallow portion. Mixed with a little shell, mica and clay pot core of soil-like - 34 cm long. Exist thin layer of shale. Shale: Sandy - Silty Shale. Mixtures of a little mica, shell and gravels core of soil-like 23-cm long.

Natural Water Content

It was found that most of the samples may have lost some of their moisture due to the length of time they were held at the site.

The laboratory tests indicate, the following range of natural water content on each sample:

Type of Soil	Range of Water Content
Clayey Silt	31.6 -36.5 %
Silty/Clayish Sand	11.3 -31.5 %
Sand/Gravelly Sand	6.9 -25.3 %
Sand and Gravel	6.7 -8.0 %
Shale	5.3 -19.6 %
"	*(10.5 -21.5 %)

*: waxed shale core.

Specific Gravity

The laboratory test indicates the following range of specific gravity on each sample:

Type of Soil	Range of Specific Gravity
Clayey Silt, Silt and Clay	2.56 - 2.61
Silty/Clayish Sand	2.57 - 2.63
Sand/Gravelly Sand	2.61 - 2.66
Sand and Gravels	2.64 - 2.68
Shale	2.61 - 2.73

Wet Density

The wet density was measured on the disturbed sampled (obtained from the split sampler and the single core tube).

The following range of wet density was observed:

Type of Soil	Wet Density (g/cm^3)
Sandy Soil	1.83 - 2.18
Shale	1.87 - 2.56

Fig. 6.3.3 SOIL PROFILE

ROUTE 1

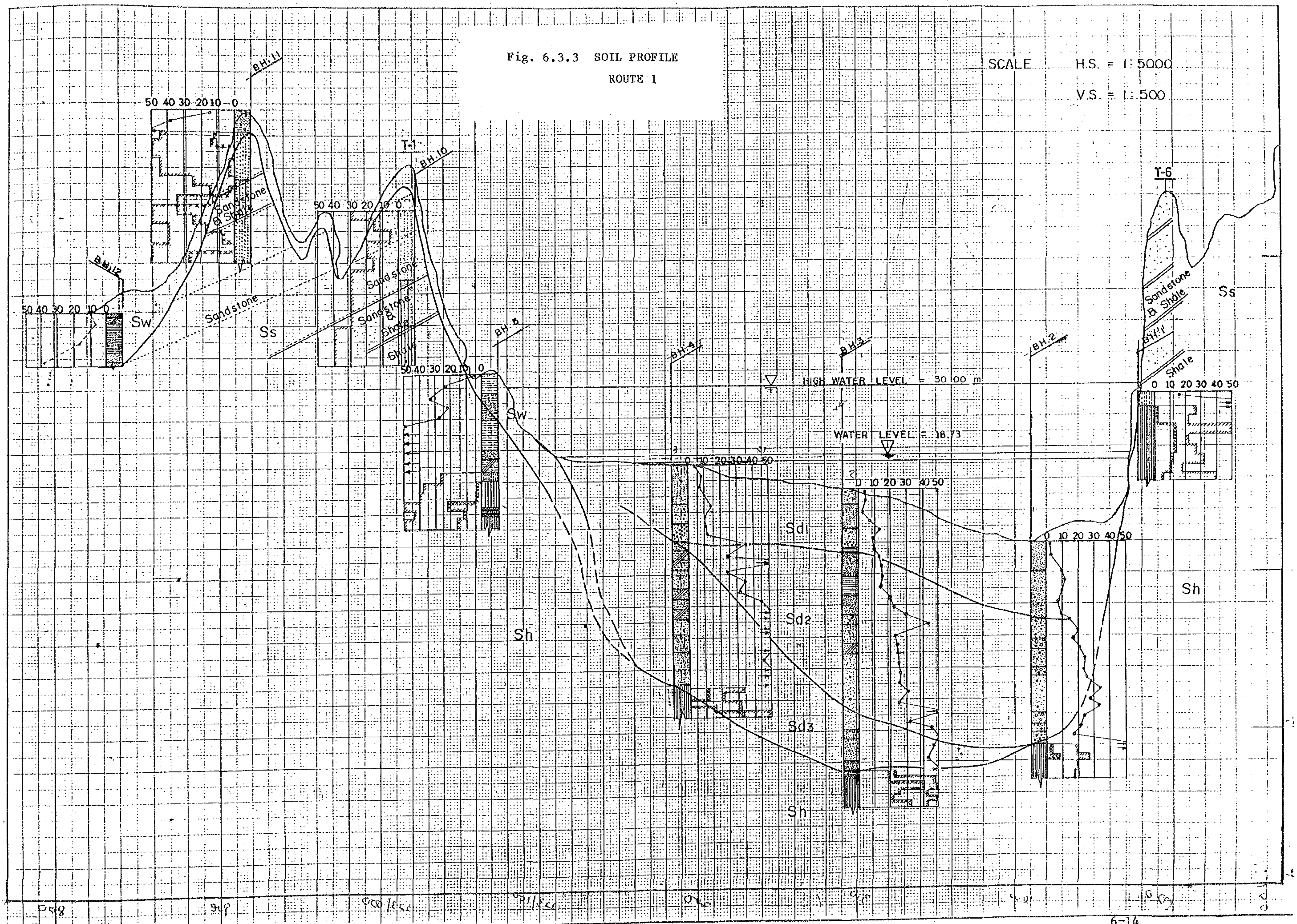
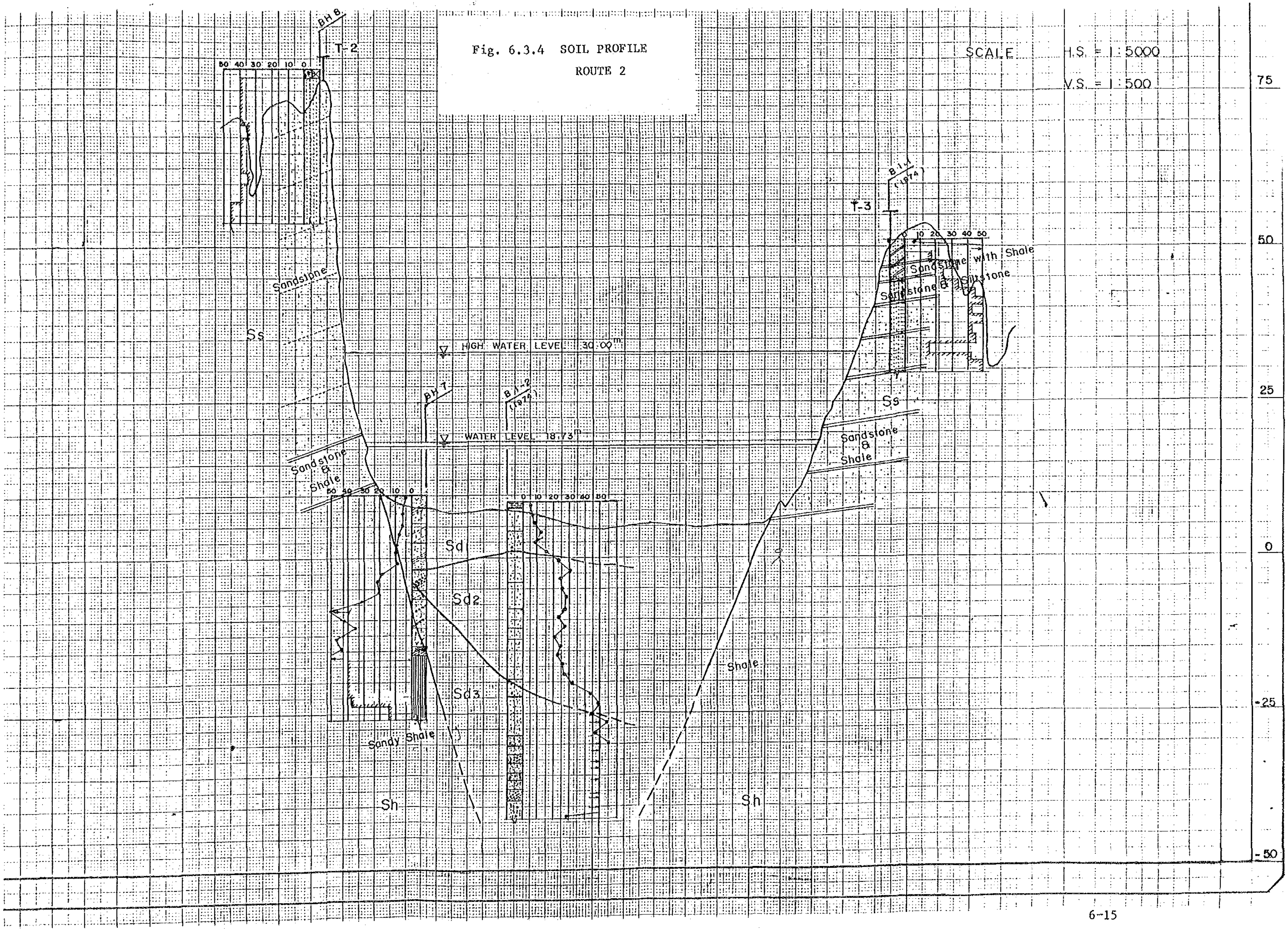


Fig. 6.3.4 SOIL PROFILE
ROUTE 2

SCALE H.S. = 1:5000
V.S. = 1:500



Atterberg Limits

The following range of the Atterberg Limits was obtained from the cohesive soils.

Type of Soil	LL (%)	PI (%)
Silt & Clay/Silty Clay	52 - 58	29 - 32
Shale	36 - 55	21 - 31

Unconfined Compression Test

The unconfined compression strength of shale core samples ranged from 3.73 kg/cm^2 to 43.8 kg/cm^2 (mainly 6 - 17 kg/cm^2).

Direct Shear Test

The direct shear test was carried out on compacted samples under quick loading conditions.

The relation between internal friction angle and N-value (where $10 \leq N \leq 38$) is shown in Appendix Fig. 6.3.47.

Compaction/CBR Tests

For the subgrade materials, three soil samples (laterite soil, cohesive soil, sandy soil) were collected from the east bank of Route 1.

The results of sieve analysis, specific gravity and Atterberg Limits are shown in the Appendix Fig. 6.3.17.

Table 6.3.3 QUANTITIES OF LABORATORY TESTS

B.H No. \ Test Item	Moisture Content	Unit Weight	Specific Gravity	Grain-size Distribution	Atterberg Limits	Unconfined Compression	Consolidation	Direct Shear	Compaction	C.B.R	
1	11	6	6	3	3						
2	27	21	13	13	1			6			
3	41	16	16	16		4		8			
4	27	4	17	13		4		6			
5	18	6	11	7	7	6					
5.A											
7											
8											
9											
(9.A)											
10											
11											
12											
Lateritic Soil			1	1					1	1	
Cohesive Soil			1	1	1				1	1	
Sand			1	1					1	1	
Total	124	53	66	55	12	14		20	3	3	

IRRAWADDY RIVER BRIDGE CONSTRUCTION PROJECT.

Fig. 6.3.5 ENGINEERING PROPERTIES OF SOILS VERSUS DEPTH

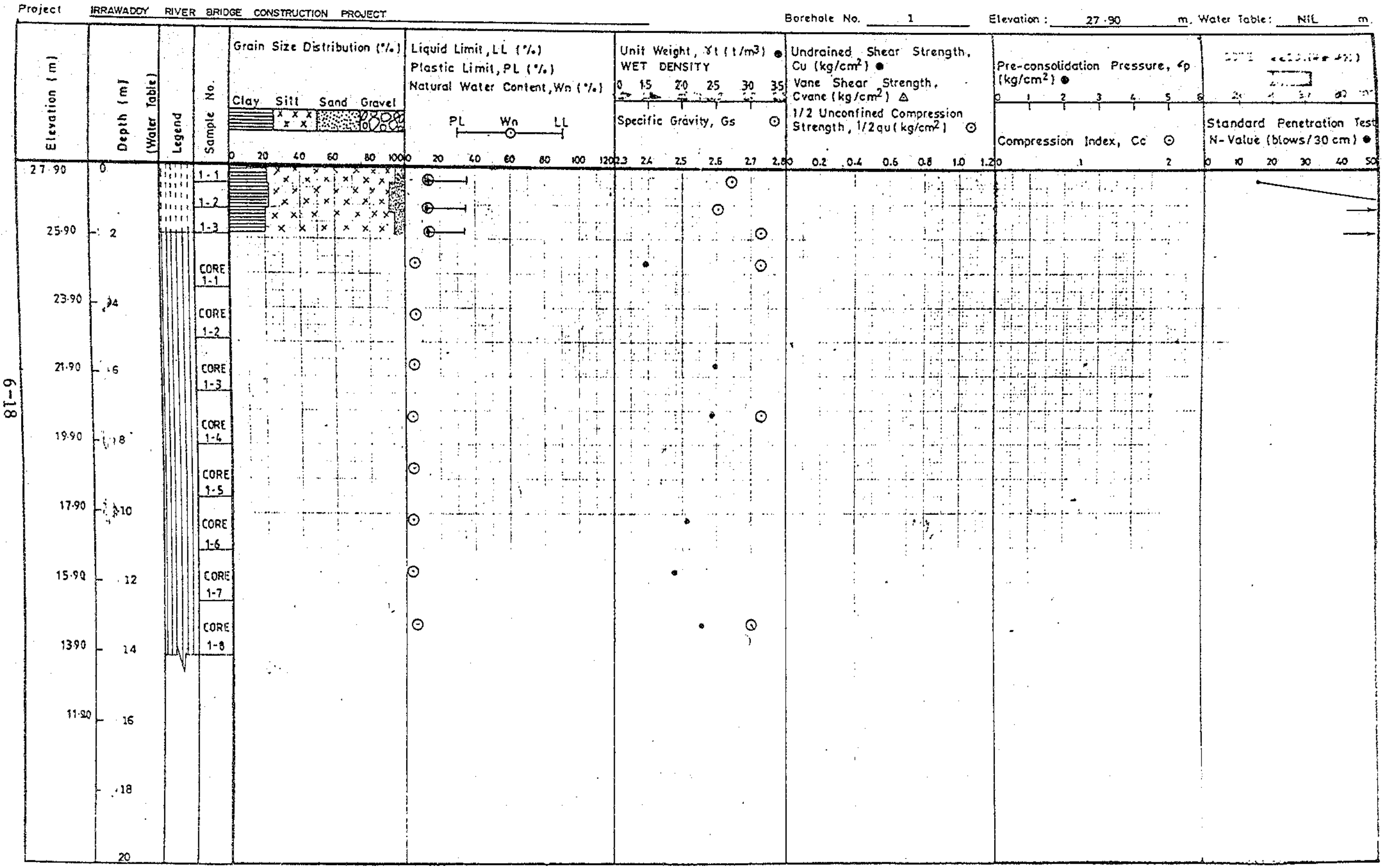


Fig. 6.3.6 ENGINEERING PROPERTIES OF SOILS VERSUS DEPTH

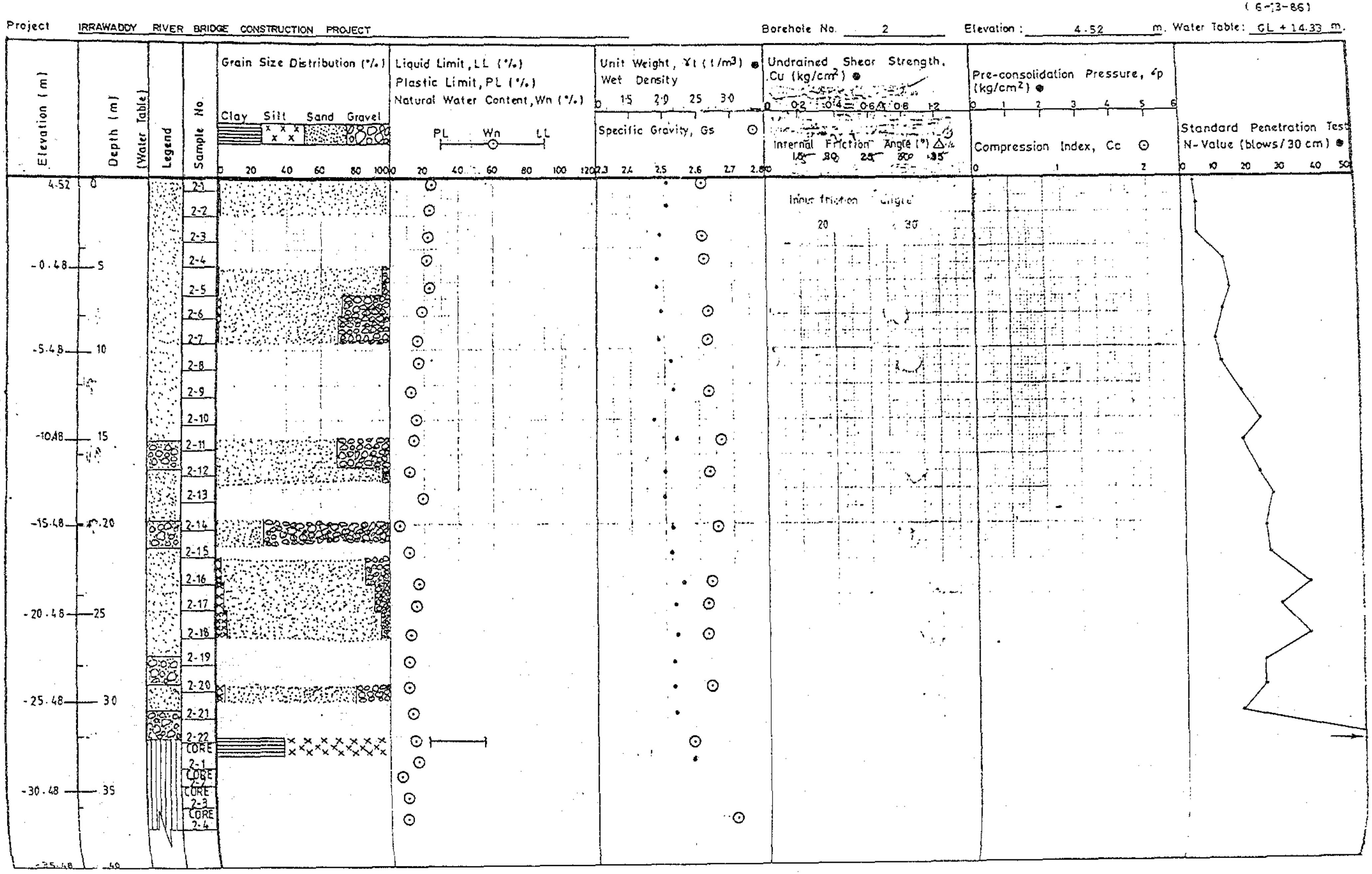


Fig. 6.3.7 ENGINEERING PROPERTIES OF SOILS VERSUS DEPTH

Project IRRAWADDY RIVER BRIDGE CONSTRUCTION PROJECT Borehole No. 5 Elevation: 32.00 m. Water Table: NIL m.

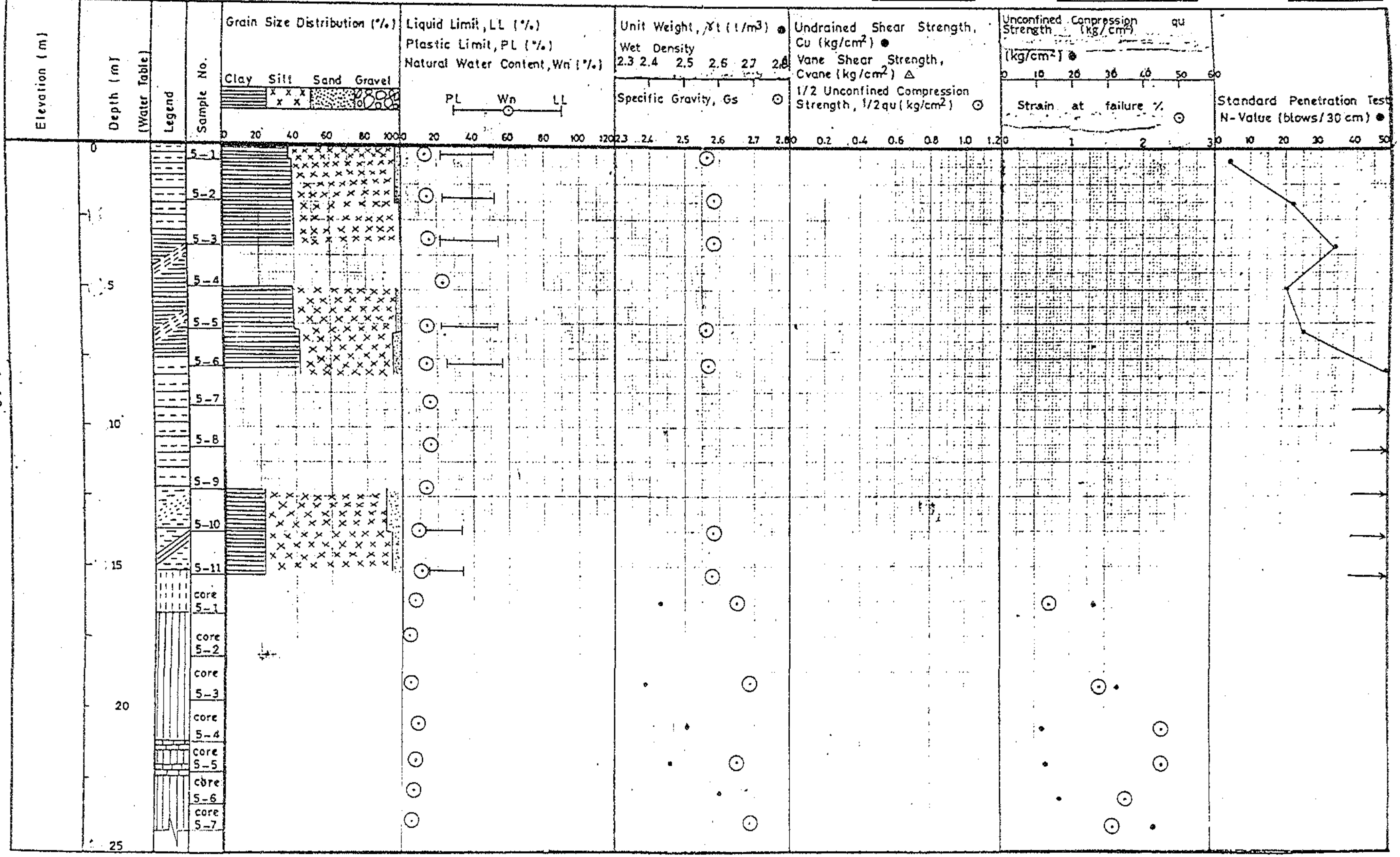


Fig. 6.3.8 ENGINEERING PROPERTIES OF SOILS VERSUS DEPTH

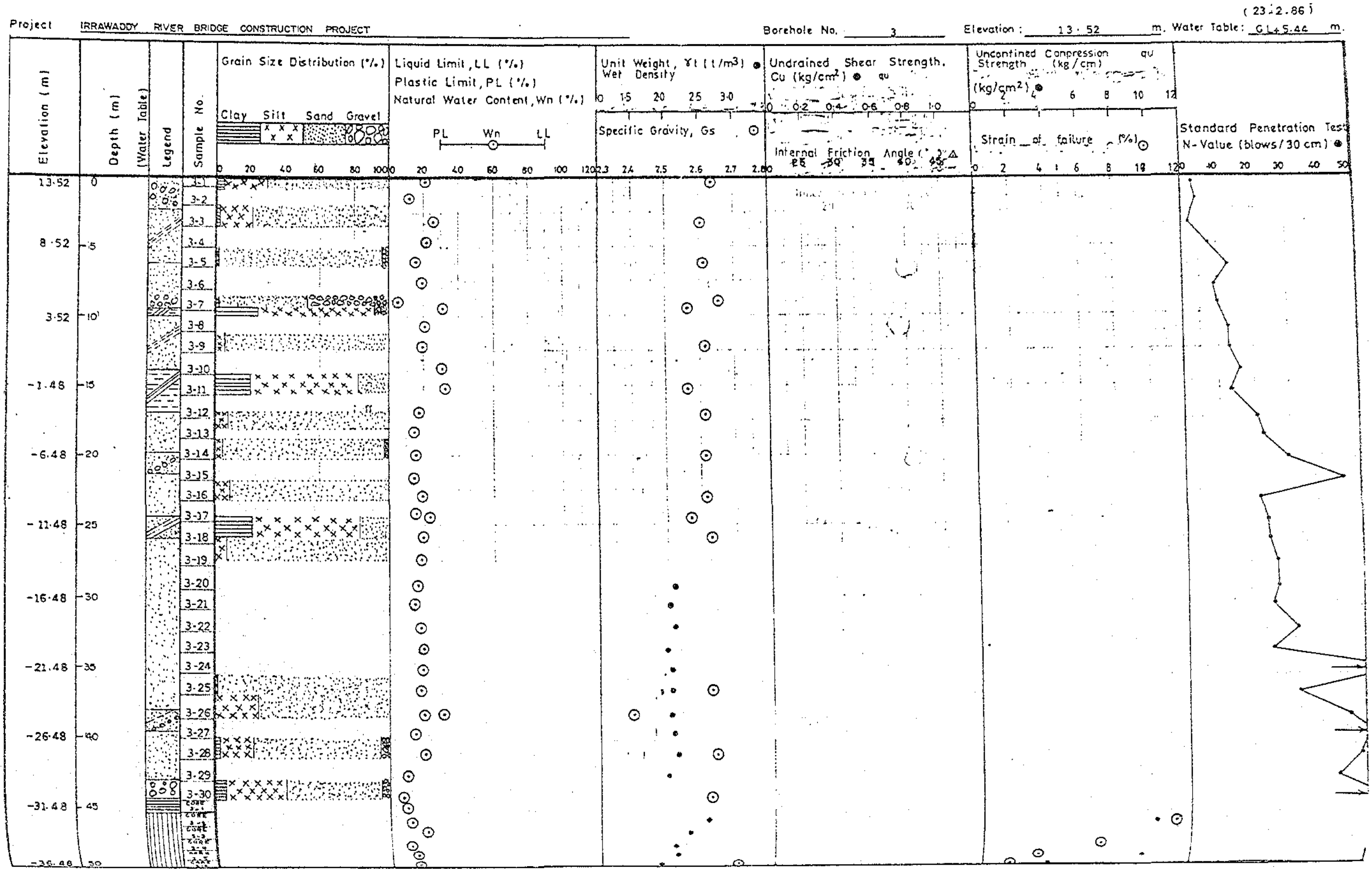


Fig. 6.3.9 ENGINEERING PROPERTIES OF SOILS VERSUS DEPTH

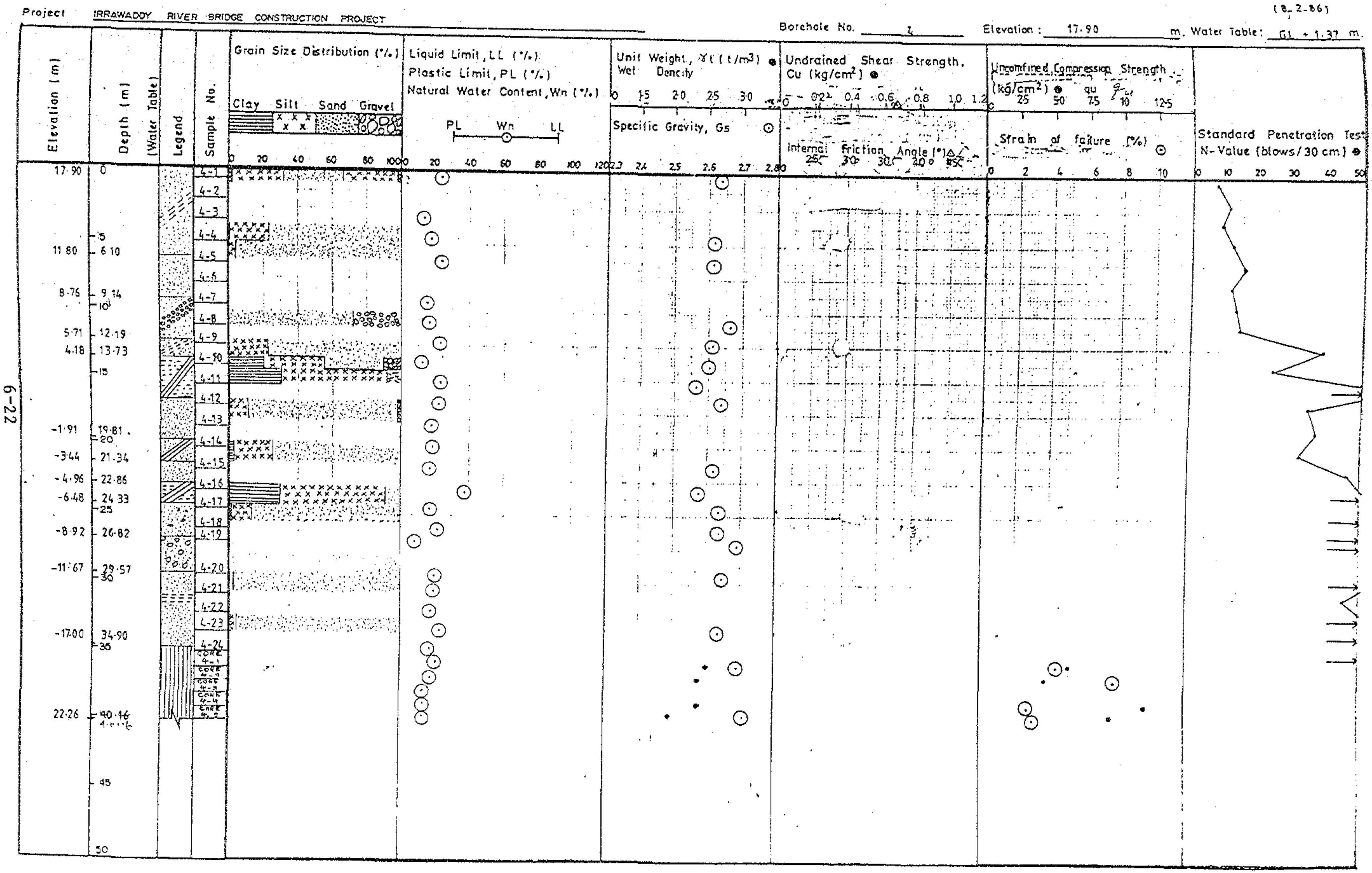
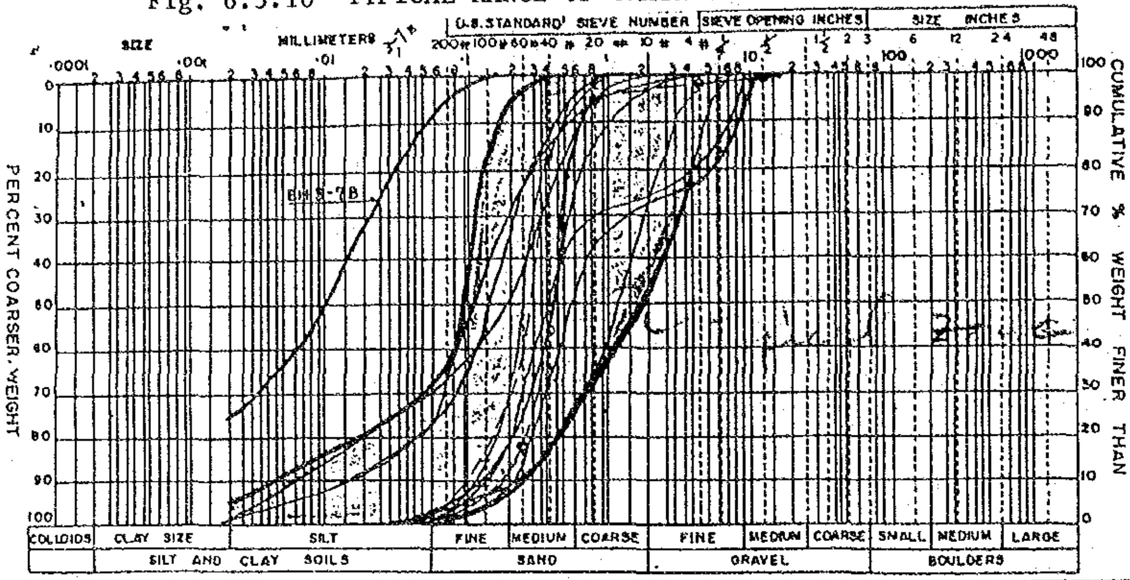
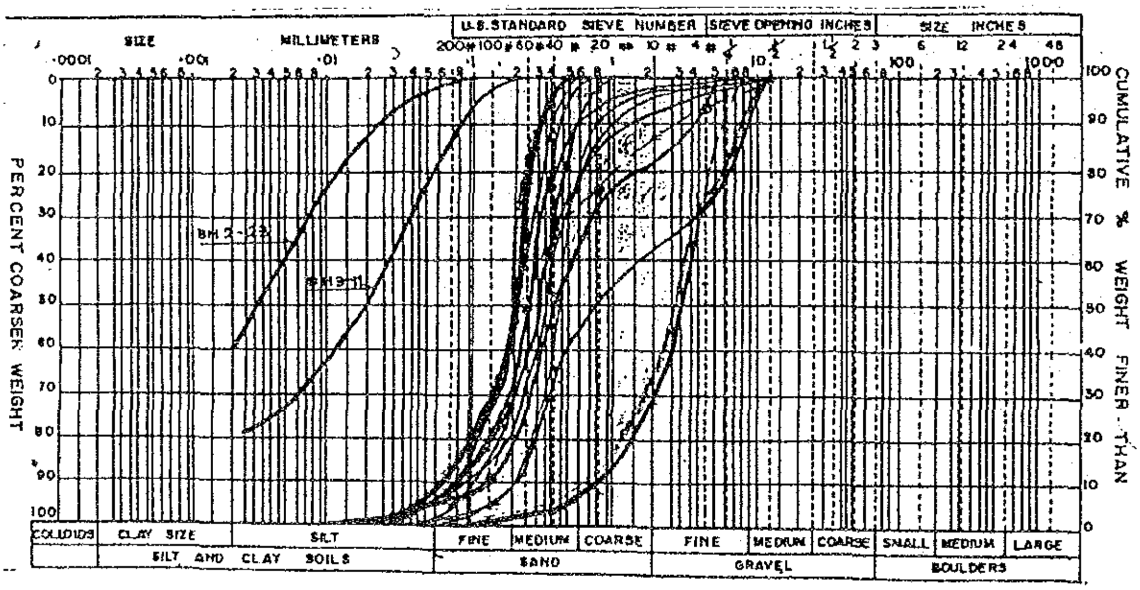


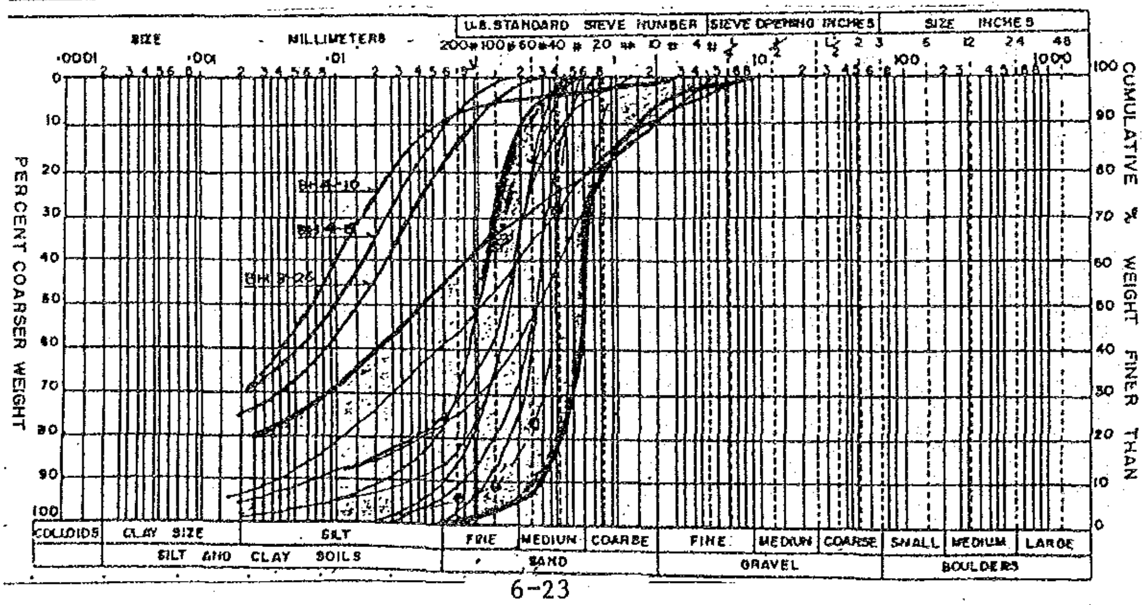
Fig. 6.3.10 TYPICAL RANGE OF GRAIN-SIZE DISTRIBUTION



Sd1



Sd2



Sd3

The modified compaction method was adopted to the compaction test, and the CBR at 95% of the maximum dry density was performed in the sample used static compaction. The test results are shown below.

	Subgrade Sample	OMC	Max. DD	CBR at 95% of Max. D
1	Laterite Soil (SM) (Gravelly Sand with Silt)	6.9%	2.11 g/cm^3	18%
2	Cohesive Soil (CL) (Clayey Silt with Sand)	10%	1.93 g/cm^3	3.5%
3	Sandy Soil (SP) (Sand with Silt)	11%	1.76 g/cm^3	8.0%

6.4 Hydrological Study

6.4.1 General

The Irrawaddy River has a catchment area of 430,000 km^2 with stream length of 1,650 km, and traverses Burma from north to south.

Annual rainfall is heavy in the upper part of the Irrawaddy River and varies from 3000 mm to 4,000 mm. In the dry zone around Mandalay and Sagaing, rainfall is about 700 mm, and 2,500 mm at Rangoon in the lower part. Annual pan-evaporation is about 1,700 mm in the middle part.

The flow of the river is mainly governed by the south-west monsoons, and the highest flood water levels occur from June to October. After October, the water level drops gradually until end of March. In April and May, depending on the melt off on the upper part, water levels are back on the rise. The average water level range at Prome is 12 - 13 meters.

Major agricultural areas of Burma are concentrated around the cities of Mandalay and Sagaing along the riverside. The river is the principal artery for transporting agricultural crops. At the upper part of the Irrawaddy and Chindwin Rivers adgradation is serious problem because of siltation. Navigation during the low water season is limited by shallow water, unstable bars, shoals, etc.

6.4.2 Meteorology

Prome maintains the nearest meteorological station to the Bridge site and the meteorological conditions are shown below.

<u>Temperature</u>

The mean maximum temperature is 102.9'F recorded in April, the minimum, 85.6F, in December. The recorded maximum temperature of 111.2'F occurred in April, 1973. The mean minimum temperature is 59.3'F in December.

Relative Humidity

The mean daily relative humidity is highest (87%) in July and August and lowest (58%) in March, with great seasonal variations, as much as 20% during the dry season.

Rainfall

Mean monthly rainfall is highest in July, approximately 250 mm. Lowest rainfall is in March. Mean rainy days per month is highest (17 days) in July and August and there are 80 days of rainfall per year.

Wind

During the rainy season, prevailing wind is predominantly from the south and south-west, because of the south-west monsoon. In the dry season the prevailing wind blows from north and north-west.

The monthly mean wind speed is around 1.6 m in April.

6.4.3 Field Investigation

(1) General

Field investigation was carried out at the Project site during 15 th to 24th in August and 22th to 24th in September.

The investigation were composed of current measurements, water level gauging, and echo-sounding.

In addition, datum levels of gauging stations of Kyawzwa, Myawaddy and Kama Lay were set out through the levelling survey. Datum levels of the gauging stations are following:

Stn.	Datum (M.S.L)
Kyawzwa	R.L = 16,1229
Myawaddy	R.L = 37,813
Kama Lay	R.L = 31,801

(2) Current Measurement

Current measurement was done along the corridor. The point velocities at the different depth (1, 3, 5, 7, 10, 15 m) were measured using TOHO Current Meter CM-1B. Recorded results are listed in Table 6.4.1.

(3) Water Level Gauging

Water levels at Kyawzwa Stn., the bridge site, and Kama Lay Stn. were gauged at regular time (9:00, 12:00, 15:00). Table 6.4.2 lists water levels gauged at the stations.

(4) Echo Sounding

For the bathymetric map of the bridge site, echo sounding survey was carried out using Tamaya Depth Finder DW-60.

The point depth from the water surface was measured at 1 minute interval. At the same time the position was observed by two control points on the west and east bank.

Table 6.4.1 CURRENT RECORDS

17-8-86 (P.M) Avg: W.L = (25.86) M.S.L

Sr. No.	CO-ORDINATE N	CO-ORDINATE E	DEPTH 1 m	3 m	5 m	7 m	10 m	15 m
1.	920654.606	2230226.058	0.80	0.90	0.85	1.00	0.95	0.95
2.	920585.403	2230355.947	1.30	1.35	1.45	1.30	1.30	1.10
3.	920758.763	2230490.842	1.20	1.05	1.20	1.20	1.20	1.10
4.	920735.892	2230621.699	1.20	1.20	1.20	1.20	1.15	1.10
5.	920780.491	2230742.581	1.00	0.95	1.00	1.00	0.95	0.95
6.	920839.956	2230854.685	1.00	1.00	1.00	0.95	0.85	0.85 (12 m)
7.	920879.053	2230990.913	0.93	0.95	0.90	Ground		
8.	920897.560	2231069.202	0.85 (1 m)	0.75 (2 m)	0.75 (3 m)	0.70 (4 m)		

18-8-86 (A.M) Avg: W.L = (25.42) M.S.L

Sr. No.	CO-ORDINATE N	CO-ORDINATE E	DEPTH 1 m	3 m	5 m	7 m	10 m	15 m
1.	920681.053	2230220.328	0.76	0.83	0.85	0.83	0.85	0.73
2.	920574.141	2230351.800	0.90	0.95	1.01	1.05	1.05	1.05
3.	920749.508	2230406.998	1.20	1.25	1.30	1.25	1.25	1.25
4.	920757.685	2230527.611	1.20	1.15	1.10	1.05	1.05	1.05
5.	920778.881	2230630.305	1.10	1.25	1.20	1.20	1.15	1.10
6.	920807.643	2230723.407	1.00	1.05	0.98	0.93	0.90	0.87
7.	920841.250	2230845.150	1.00	1.00	0.95	0.95	0.90	
8.	920870.764	2230963.383	0.95	0.93	0.83	0.78		
9.	920887.707	2231024.872	0.91	0.83	0.79			
10.	920908.544	2231075.053	0.85 (1 m)	0.78 (2 m)	0.74 (3 m)	0.73 (4 m)	0.72 (5 m)	

23-9-86 Avg: W.L = (29.66) M.S.L

Sr: NO.	TIME	CO-ORDINATE N	CO-ORDINATE E	DEPTH 1 m	3 m	5 m	7 m	10 m
1	9:56	920664.7214	2230231.476	1.80	1.50	1.30	1.30	1.30
2	10:34	920873.4169	2230849.430	1.50	1.50	1.45	1.40	1.20
3	10:47	920880.3923	2231009.234	1.40	1.40	1.40	1.40	1.30
4	11:05	920795.3728	2230797.662	1.30	1.30	1.25	1.50	1.50
5	11:34	920667.3838	2230509.050	1.70	1.70	1.70	1.70	1.70

Study Team

Table 6.4.2 WATER LEVEL GAUGING RECORD

Stn	Kyawzwa Gau.			Bridge Site			Kama Lay Gau.		
Day \ Time	9:00	12:00	15:00	9:00	12:00	15:00	9:00	12:00	15:00
17 Aug	25.729	25.650	25.574			25.698		25.917	25.856
18	25.169	25.096	25.053	25.277	25.208	25.174	25.429	25.399	25.338
19	24.837	24.799	24.761	24.929	24.890	24.852	25.094	25.078	25.063
20	24.711	24.711	24.711	24.800	24.796	24.879	25.002	24.972	25.002
21	24.787	24.787	24.812	24.866	24.879	24.899	25.033	25.094	25.094
22	24.914	24.939	24.952	24.993	25.020	25.029	25.185	25.216	25.246
23	25.092	25.117	25.140	25.200	25.213	25.259	25.399	25.429	25.460
22 Sep	29.340	29.380	29.430	29.450	29.510	29.546			
23	29.534	29.534	29.534	29.656	29.657	29.659			
24	29.430	29.430	29.407	29.576	29.565	29.563			

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6.4.4 Hydrological Stations

There are more than 30 hydrological stations installed in the Irrawaddy River Basin. Near the Project site, 4 stations (Prome, Kyawzwa, Myawaddy and Kama Lay) is available. Fig. 6.4.1 shows hydrological stations in Burma.

(1) Prome Station

The stations is situated 20 km downstream of the bridge site. As a primary station in the middle catchment of the Irrawaddy Basin it has played an important role in flood forecasting. The water level is gauged at regular time (6:30, 12:30, 18:30). Data obtained here is compiled and kept at the Headquarters of the Department of Meteorology and Hydrology.

(2) Kyawzwa Station

The station was established 450 m north of Kyawzwa, 1,530 m downstream of the bridge site on May 1978 for the No. 3 Fertilizer Plant Project. Here, water level, current velocity, sediment discharge and temperature were measured. In addition, cross-section survey and current velocity gauging were done for the rating curve at this station.

(3) Myawaddy Station

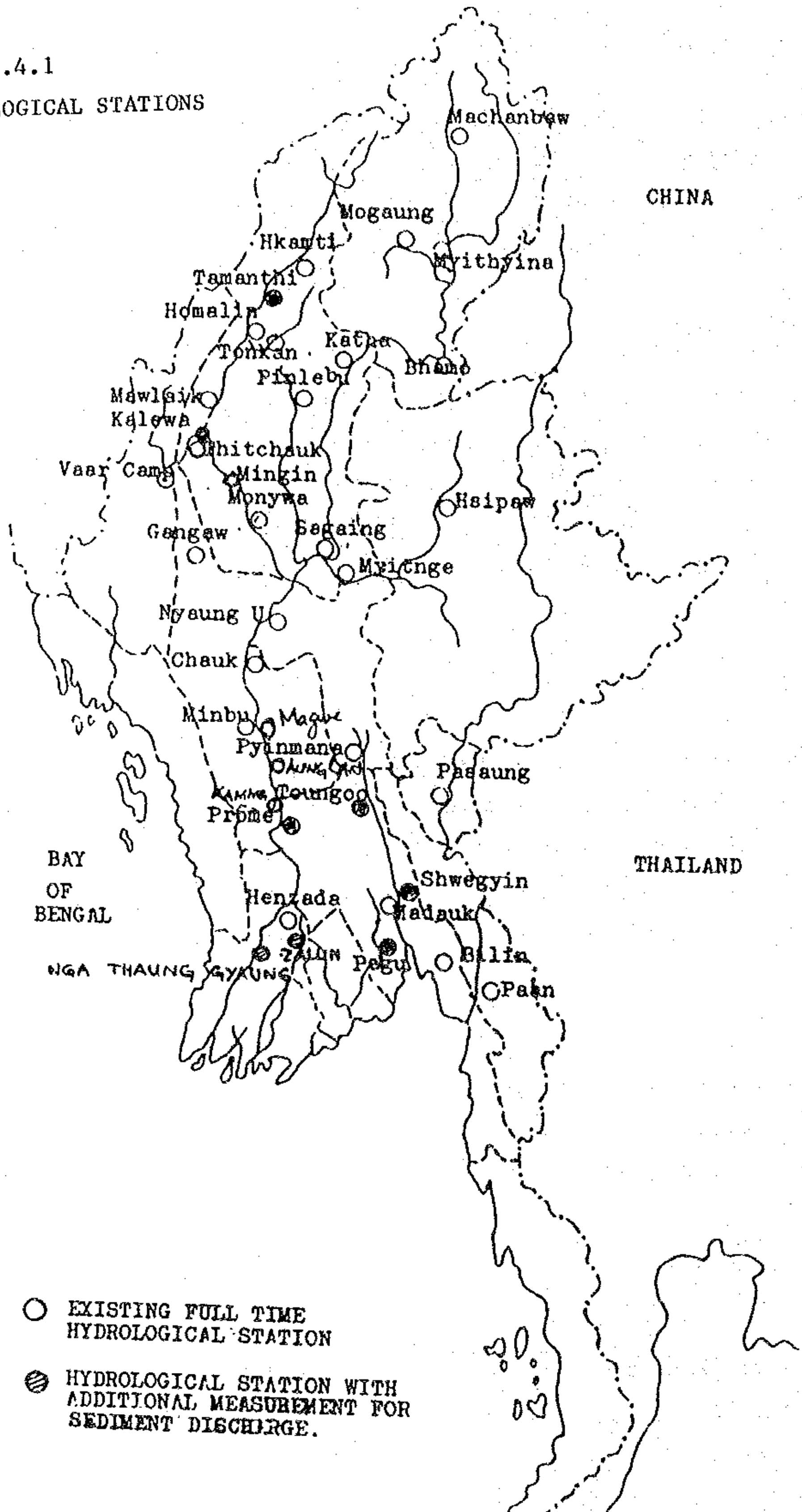
Construction Corporation established this station at the West bank in the vicinity of the bridge site for this project on October 1984. Daily water level record is available from Oct. 1984 to Dec. 1985.

(4) Kama Lay Station

The station is situated at a bend of 2 km upstream from the bridge site. This station also was constructed by C.C for this bridge project. Datum level of this gauging were set out in this field investigation.

Fig. 6.4.1

HYDROLOGICAL STATIONS



Source: MHD

6.4.5 Water Level

(1) Water Level Correlation

Correlation analysis between Prome Stn. and Myawaddy Gauging Stn. was tried using daily average water levels from October to November 1985.

Water level response of 2 stations is highly good and the correlation coefficient is 0.997.

Regression formula for conversion Prome Stn. value to Myawaddy's value is as the following:

$$Hm = 1.1355\ Hp - 0.8344$$

Hm: Water Level at Myawaddy Stn.

Hp: Water Level at Prome Stn.

(2) Water Level Regime

Based on the regression formula, water level regime at Myawaddy were derived from Prome's water level of recent 5 years as following. Appendix Table 6.4.1 lists water levels at Prome in recent 5 years.

WATER LEVEL REGIME AT THE SITE (m. M.S.L)

	Hmax	H(95)	H(185)	H(275)	H(355)	H(min.)	Hmean
YEAR; 1979	31.63	26.52	21.01	19.17	18.44	18.35	22.71
YEAR; 1980	31.47	27.06	21.38	20.00	19.05	18.93	23.49
YEAR; 1981	30.42	25.15	20.86	19.50	18.83	18.57	22.80
YEAR; 1982	31.18	27.78	20.24	19.10	18.33	18.19	22.66
YEAR; 1983	31.60	27.02	22.09	20.17	18.76	18.56	23.43

Hmax: Maximum Water Level

H(N-): N-th Water Level from the Maximum

(3) Flood Water Level

(a) Bridge Site

Bench marks for the past flood water level were established at the bridge site. Flood water levels of these marks are shown below.

Flood Year	Water Level (M.S.L)
1974	107.83 (ft) 32.87 (m)
1984	103.25 (ft) 31.47 (m)

Flood water level has never threatened the homes of those on the East bank.

(b) Prome Station

Yearly extreme water levels compiled since 1868 are listed in Appendix Table 6.4.2.

Maximum flood level was 103.1 feet (31.42 m M.S.L) in 1948. 1974 flood level was 30.24 m M.S.L, 2.6 m lower than the level at the bridge site.

(c) Probable Flood Water Level

Using 96 extreme flood levels at Prome Stn., probable flood levels are estimated by assuming log-normal distribution. Thomas method and Iwai method indicates approximately similar values as following.

Probable Flood Water Level at Prome

(EL.m M.S.L)

Return Period	5	10	20	30	50	100
THOMAS	29.3	29.6	29.9	30.2	30.2	30.5
IWAI	29.0	29.6	29.9	30.2	30.2	30.5

According to the above values, the 1974 flood level corresponds to the 50 year return period.

(4) Design Water Level

Design water level (D.W.L) for this bridge planning was estimated at EL. 33.00 M.S.L including allowance based on ever recorded maximum water level (EL 32.87) at the bridge site. D.W.L corresponds to the 100 year water level.

(5) Water Surface Profile

Water levels gauged at Kyawzwa Stn. and the bridge site are listed in Table 6.4.2.

The difference of water level was approximately 0.11 m, and the profile was 1/14,000 gradient.

(6) Rating Curve

Rating curve at Kyawzwa was based on 46 measurements between May 1978 and November 1979, and the water stage-discharge relationship of $Q = 299.02\ (H-15.00)^{1.726}$ was derived from the rating curve.

At the water level gauged on 17th August (W.L = 25.86) and 23th September (W.L = 29.66), accumlated discharge was assumed 15,000 and 27,000 m^3 per second. These value were around 10 - 15% fewer than values derived from the formula with consideration of the water level difference between two gauging sites.

6.4.6 Scouring

Local scouring depth arround the bridge foundation was calculated using empirical methods. Numerous equations have been developed to predict the scouring depth in cohesionless bed adjacent to a cylindrical pier.

Equations of Shen, Laursen, Blench, Andru and Pona were used for the calculation. Bridge piers in this project should be founded on the groups of piles. A conservation estimate may be obtained by assuming that the width of outermost pile in the group is adopted as the width of a single pier in these calculation. Of these equations, Andru's formula is based on available experimental and field data gathered in India and Pakistan. Velocity distributions, median particle size and bed level around the pier obtained through the field investigation were considered for the calculation.

(a) Shen I

$$Ds = 1.17 \times (Uob)^{0.62}$$

(b) Shen II

$$Ds = 1.59 \times (Uob)^{0.67}$$

(c) Laursen

$$Ds = 1.11 \times (Yob)^{0.5}$$

(d) Blench

$$Ds = 1.8 * Yo^{0.5} * b^{0.25} - Yo$$

(e) Andru

$$(Dsf + ho)\ Fb^{1/3} = 1.8 \cdot q^{2/3}$$

(f) Inglish - Pona

$$Dsf = 1.7b\left(\frac{q^{2/3}}{b}\right)^{0.78}$$

Here,

Ds = Scouring Depth (m)

Dsf = Scouring depth below mean bed level (feet)

Uo = Approach Velocity (m/s)

b = Width of Pier (m)

Yo = $0.38 \times q^{0.67}\ D_{50}$

qo = Discharge per Unit Width of a Pier (m^2/s)

D_{50} = Median Particle Size (0.4 mm)

ho = Mean Depth above mean bed level (feet)

Fb = Uo^2/ho

The calculated values arround No. 6 Pier in the center and No. 8 Pier near the west bank are shown below, and recommendatory scouring depth with allowance for the engineering is 12 m around deeper zone near the west bank and 10 m around the center.

Scouring Depth (m)

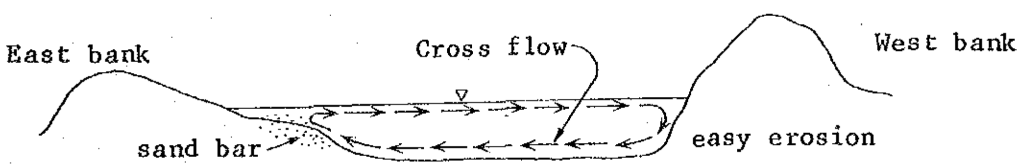
Formula	Location: No 8 Pier	No 6 Pier
Shen I	8.7	8.1
Shen II	13.9	12.8
Laursen	8.5	6.1
Blench	5.9	4.0
Average	9.3	7.8
Andru	6.6	6.6
Inglish-Pona	7.9	7.9

6.4.7 River Stability

Channel at the bridge site is almost straight, but the channel near Town Kama of 2 km upstream is bending.

In the east bank, brown cohesive sedimentary soils predominate and flood plain can't be seen. In the west bank, bank slope is composed of grayish brown fine to medium grained stone alternating with gray silty shale. Outcrops of sandstone remains in steep river bank. River bed is covered with brown or dark gray sandy soil. Moving alternating bars couldn't be seen in channel.

In general, moving alternating bars are formed sometime near either bank side by turn in the straight channel as this. Alternating bars move downstream to change their cross section shape. In law water season, changing bars, shoals, shallow and deep pools appear to restrain navigation. As hydraulic characteristics of bending channel, cross flow occur toward outside by centrifugal force. Flow erodes the outside bank to migrate the channel as shown in typical sketch.



Cross flow in downstream of bending channel

Through examining available records and field inspection, the following points are identified:

(1) Sandstone outcrops of steep slope in the west bank prevent the bank erosion and migration of the channel and channel spray couldn't be seen.

(2) When compared the cross-sections obtained by echo-sounding survey in dry and wet season (Appendix Table 6.4.3), the difference of cross-section area at Design High Water Level was few (5%) and the shape was almost similar.

(3) Water level profile at high water season was mild, 1/14,000 gradient. Accordingly, alternating bars should be difficult to be formed.

(4) Bathymetric maps of dry and wet seasons indicates that deepest zone near the west bank is almost still.

(5) Judged from the water level regime, river bed is covered with flow during 200 days a year. The sand bar near the east bank is not moving.

Based on above findings, channel at the site is statically stable and the main stream course is still and unchangeble.

CHAPTER 7 ENGINEERING STUDY

CHAPTER 7 ENGINEERING STUDY

7.1 General

The area in which the bridge location is examined has the following characteristics.

- Over a distance of 550 miles from the river mouth to the confluence with the Chindwin River, the location near Myawaddy/Kyawzwa is the mid-point of the Irrawaddy River.
- Prome (the largest town in the direct influence area) is the regional center of transport network on the east side area: roads to Rangoon, Mandalay and through Pegu Yoma to Oktwin and railways to Rangoon and a planned line to Taungdwingyi joining in the existing Mandalay - Rangoon system. The bridge's proximity to Prome has advantageous approach to these trunk lines.
- The bridge is located between the industrial town of Padaung and crude oil and natural gas production area of Minbu town. The bridge will support the development of these industrial growth cores by linking them by railways/roads.
- Scope of work defined the study's examination area within 3 miles along the river arround Myawaddy village, in which route study is conducted as in the following sections.

Under the collected data in Phase I, engineering study of this chapter was conducted mainly for the examination of design standards and criteria, for investigation of appricable design concept, for the selection of alternative bridge location and for the selection of alternative span arrangements and the selection of alternative bridge types.

7.2 Design Standards and Criteria

7.2.1 *Basic Conditions*

The design standards and criteria for the project are governed by the details given in the standards currently in use in Burma, and standards used in Japan are referred to when necessary.

In this section the main elements of adopted standards and the criteria which give fundamental effects to Phase I Study, including cross sections, are presented, and will be further refined in Chapter 8.

7.2.2 Geometric Design Standards

The geometric design standards for the project have been selected in accordance with the policy on geometric design standards for roads and construction standards for railways in Burma. The main elements of the adopted standards for each feature are as follows:-

a) Road Design

- Design Speed - 96.6 km/h (60 m /h)
- Carriageway - 8.5 m (28 ft)

In the case of the bridge, the carriageway is 8.5m (28 ft) or 4.5m (15 ft) curb to curb depending on whether it is two-way or one-way.

- Shoulder - 1.8 m (6 ft)
- Side Walk Width - 1.5 m (5 ft)
- Maximum Longitudinal Grade - 3%

- Superelevation

Bridge	-	1.5% (on both roadway and sidewalk)
Approach Roads	-	2.0%

- Construction Gauge - as shown in Fig. 7.2.3.3

b) Railway Design

- Maximum Speed - 96.6 m/h (60 ml/h)
- Maximum Degree of Curve - 6° (radius 291m)
- Maximum Longitudinal Grade - 1%
- Construction Gauge - as shown in Fig. 7.2.3.4

c) Navigation Clearance

- Minimum Vertical Clerance above the Highest Water Level - 17 m (55 ft)
- Minimum Horizontal Clearance - 107 m (350 ft)

Four navigational channels, each width of 107 m, are to be prepared.

7.2.3 Loading and Codes

The design of the bridge structure is made with reference to the following documents:-

- Japanese Specification for Highway Bridge (JSHB)
- Structure Design Standards by the Japanese National Railways (SDJNR)
- Superstructure Design Standards by Honshu-Shikoku Bridge (SDSHS)
- Indian Railway Standards (IRS)

Fig. 7.2.3.1 60-T TRAILER LOAD

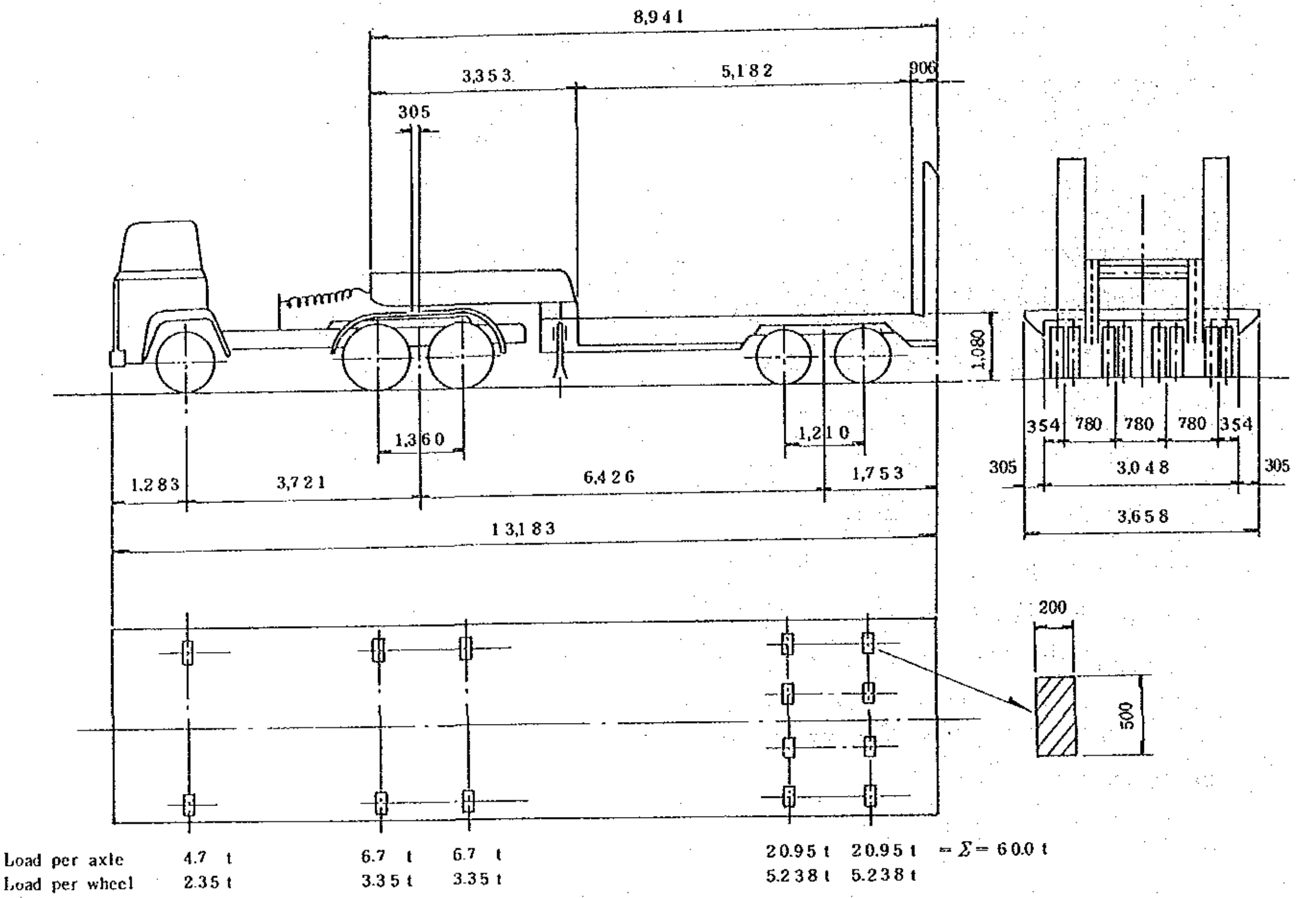


Fig. 7.2.3.2 STANDARD LOADING DIAGRAM

M.L

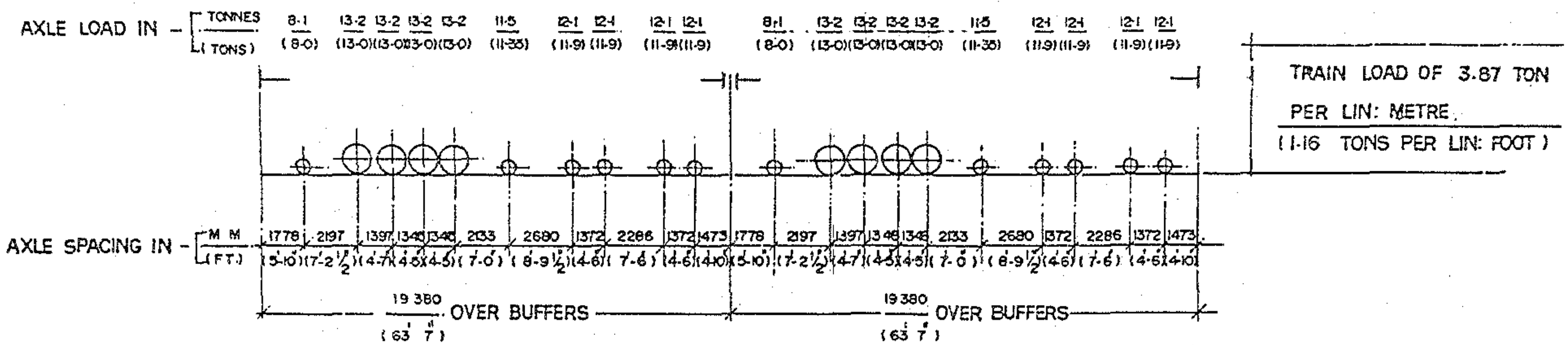


Fig. 7.2.3.3 CONSTRUCTION GAUGE OF ROAD

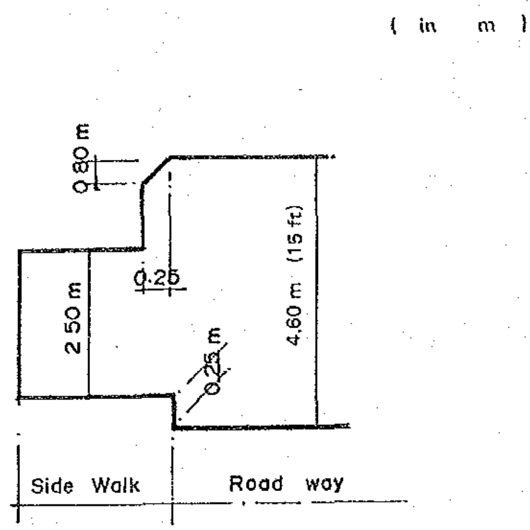
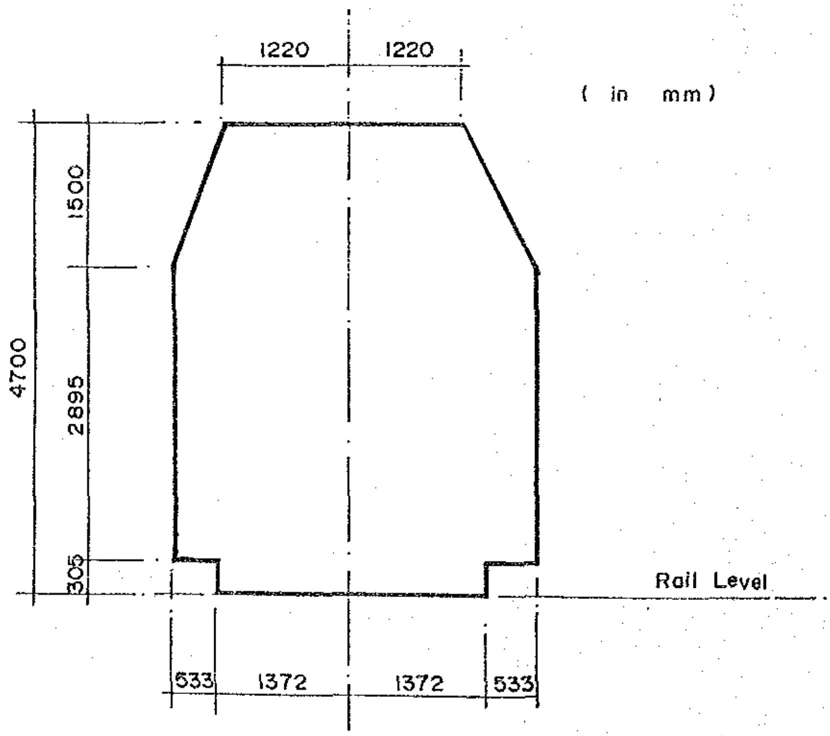


Fig. 7.2.3.4 CONSTRUCTION GAUGE OF RAILWAY



The main factors of the adopted standards for each feature are as follows:

- Live Load

  For the road, TL-20 specified in JSHB is adopted as the live load standard. This applies a 60 ton truck load as shown in Fig. 7.2.3.1 to check the strength of the slabs and floor systems, in which the pay load is reduced to 60 ton total weight.

  For the railway, standard loading diagram shown in Fig. 7.2.3.2 are adopted using impact factors specified in SDSHS.

- Effect of Earthquake

  Providing that the Modified Seismic Co-efficient Method is applied for estimating the effects of earthquakes, the standard Horizontal Design Seismic Co-efficient of 0.1 is adopted. (modified horizontal design seismic coefficient $K_h = 0.125$)

7.2.4 Cross Section

The road bridge has only one type of cross section, i.e., sidewalk + carriageway + sidewalk, but the rail-cum-road bridge has several alternative configurations.

The alternative configurations of rail-cum-road bridge are divided into the following three groups depending on traffic function:

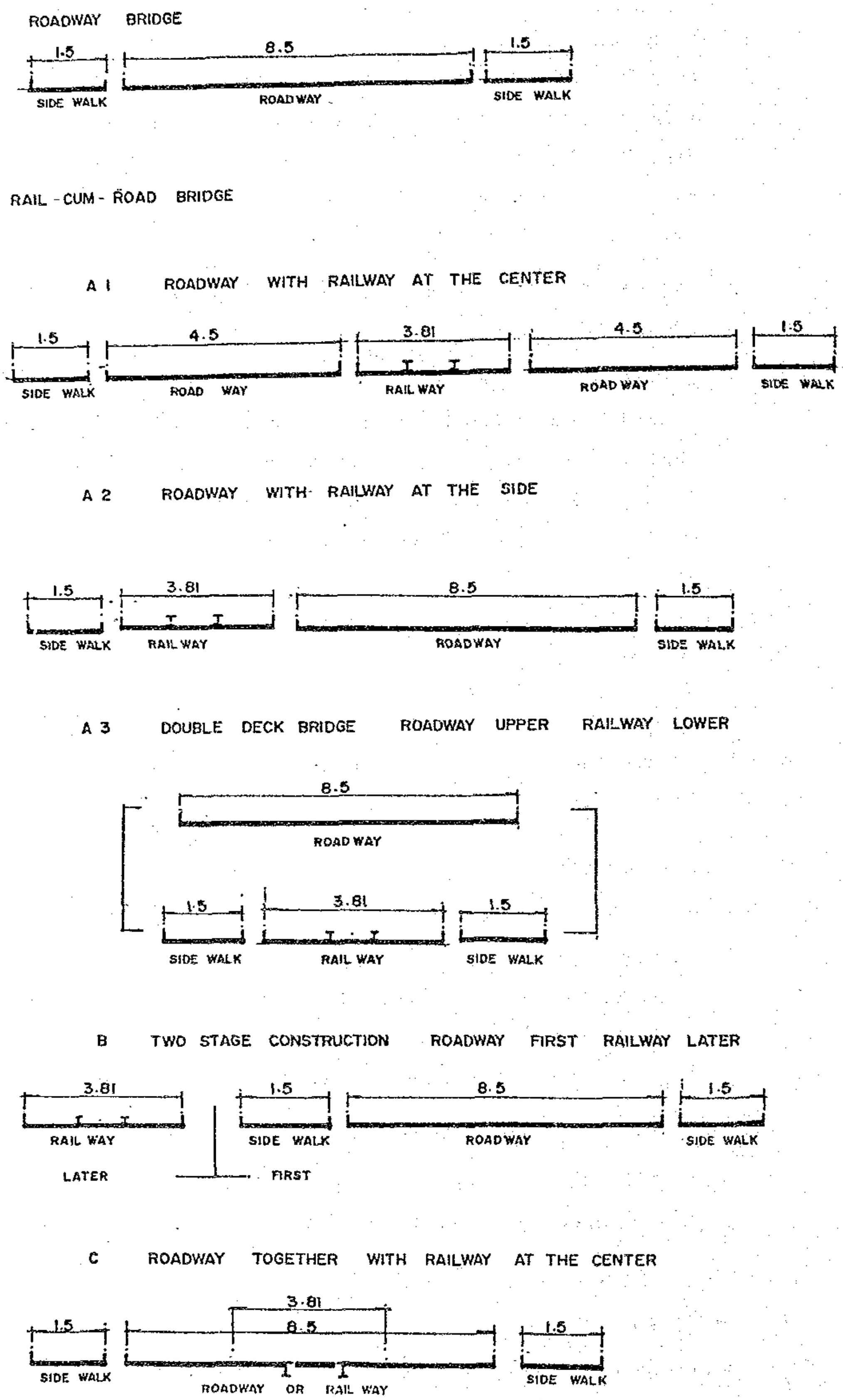
A. Simultaneous traffic movement on roadway and railway
   - Roadway with railway at the center
   - Roadway with railway at the side
   - Double deck bridge: roadway upper, railway lower.

B. Two-stage construction: roadway first, railway later.

C. Alternating traffic between roadway and railway.
   - Roadway and railway combined within one carriageway. Road traffic stops to allow trains to cross the bridge.

Diagrams of these cross sections are shown in Fig. 7.2.4.1. Cross sections are drawn from the west bank forward the east.

Fig. 7.2.4.1 CROSS SECTIONS OF BRIDGE



Among the first alternatives the alternative of simultaneous traffic movement, "Roadway with railway at the center" has the following disadvantages compared with the other two alternatives in the same traffic group:

- When traffic accident occurs on the road, it is difficult to divert the vehicles stopped by the accident.
- At both approaches to the bridge, the railway has to cross the roadway.

From these disadvantages the "Roadway with railway at the center" is deleted from further discussion of this report.

7.3 Conceptual Design of Bridges

7.3.1 General

For the proposed bridge at the established alternative routes discussed in section 7.3.5., a preliminary study was undertaken on span length, varying from 50 m to 200 m to identify suitable span length using appropriate types of bridge structure. It showed that a bridge with a span length of approximately 100 meters has the lowest construction cost.

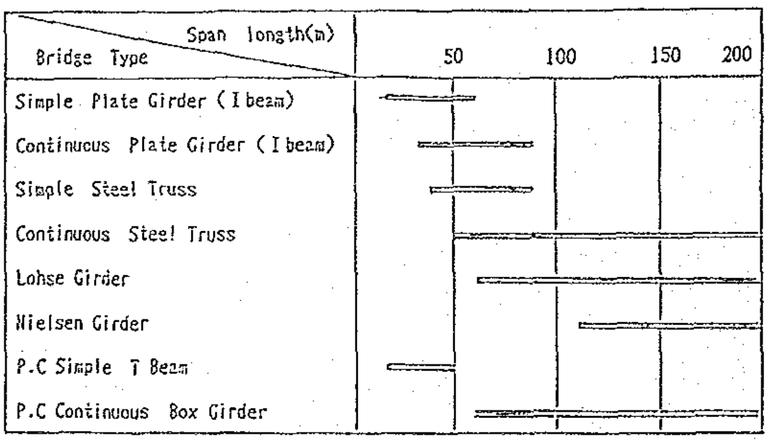
In this section, the structural types of bridge with a span length of about 100 meters are studied.

7.3.2 Superstructure

Applicable span ranges of superstructure types defined by past experience are shown in Fig. 7.3.2.1. From this figure with respect to a 100 m span length, a PC continuous structure girder is appropriate for a concrete bridge, while continuous truss is appropriate for a steel bridge.

For these structural type alternatives, the construction would not be disturbed by the flood in the rainy season as it can be built adopting the cantilever erection method without using any scaffolding in the river.

Fig. 7.3.2.1 STANDARD BRIDGE TYPES AND SPAN LENGTH



7.3.3 Substructure

The large diameter cast-in-place concrete pile and the caisson foundation are considered appropriate foundations for the span arrangement of the proposed bridge, when bridge site conditions as shown in Table 7.4.2.1 are taken into account.

The pile foundation including footings can be constructed in the six months of dry season to form one complete unit of the foundation, but the caisson would require nearly one year. Either the island method or the pre-cast method can be applied for a caisson but both can be utilized only in the dry season. As the construction term has to overlap at least two seasons, the caisson, once in the construction stage, would encounter difficulties caused by floods during the rainy season and is not deemed an appropriate construction practice.

On the other hand, our preliminary structural analysis indicates that the pile foundation not only ensures safety during the course of the construction, but is also secure against possible scouring for a long period of time. Because of this, the cast-in-place large diameter concrete pile (R.C.D. Method) is considered more appropriate as the foundation structure for this bridge.

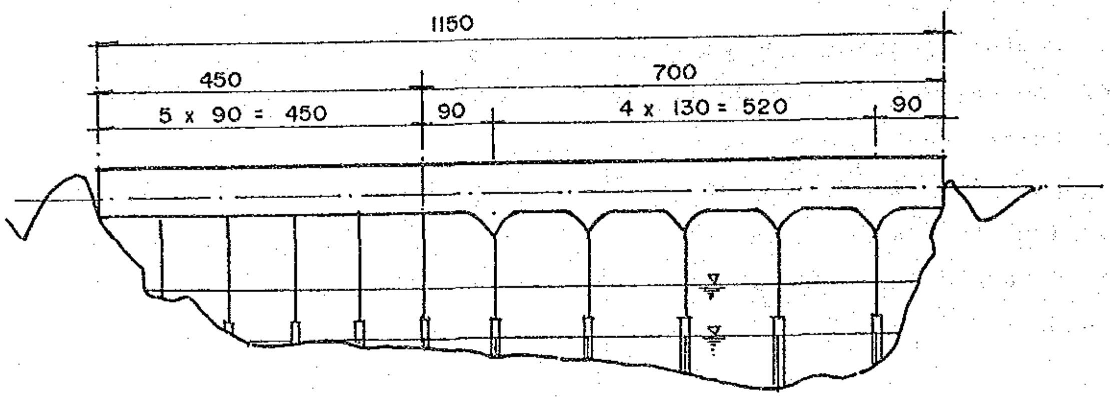
7.3.4 Span Arrangement

Span arrangements for alternatives of Route I and Route II selected in paragraph 7.4 are shown in Fig. 7.3.4.1. The total of 700 m covering the navigational channels is determined by four channels with the required width of a channel at 130 m (the navigation channel of 107 m plus the width of the pile cap), dimensions of the substructure, and physical characteristics of the superstructure. There is no alternative arrangement over these channels.

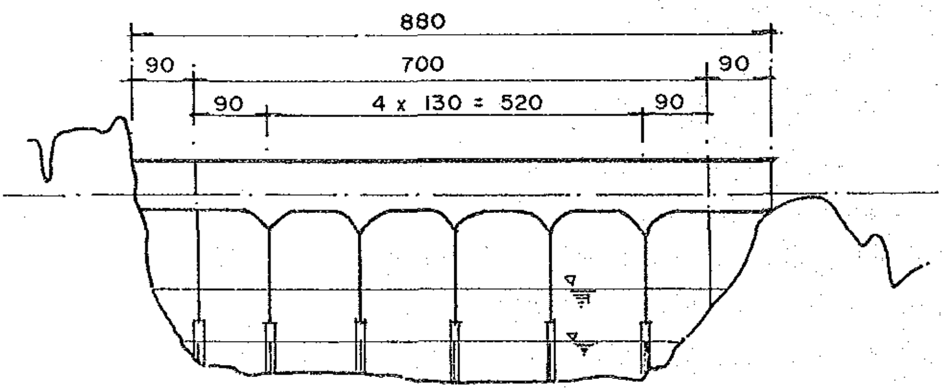
Over the shallows excluding the above navigation channels the followings are identified: on the Route II there is no other plan than the span arrangement shown in Fig. 7.3.4.1 because of the river width and the bridge length (880 m) as well as the cross section of the river bed, and on the Route I it can contain some alternative ideas for the

Fig. 7.3.4.1 SPAN ARRANGEMENT

Route No. 1



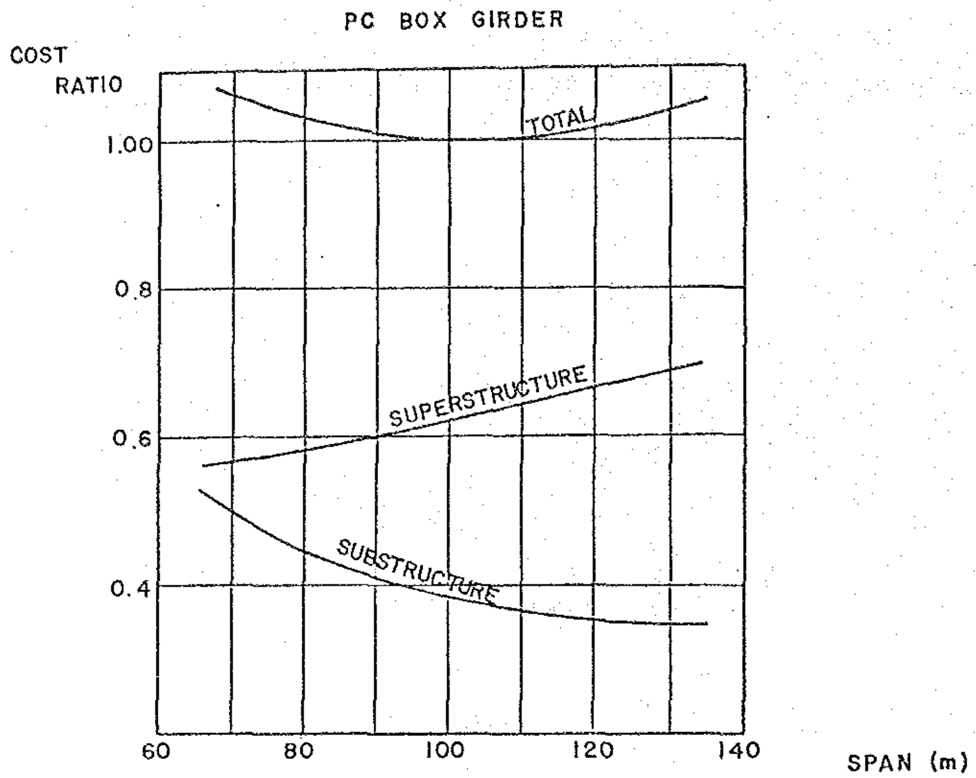
Route No. 2

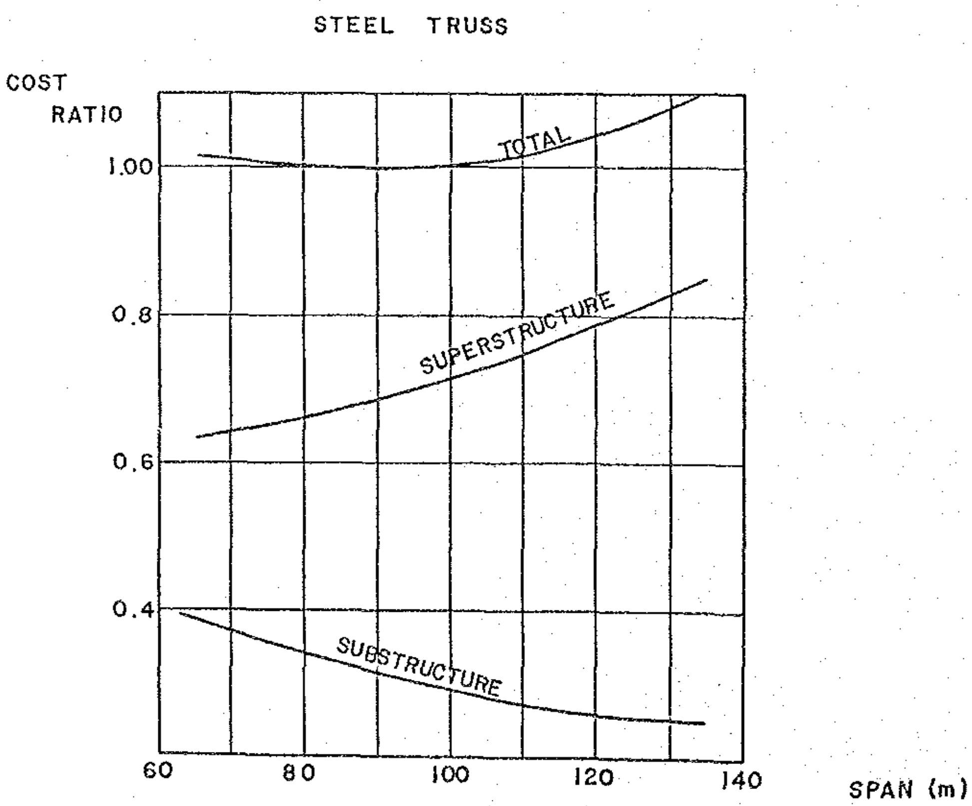


spanning over 450 m because of the bridge length 1,150 m and the navigation channel width 700 m.

The outcome of the study shown in Fig. 7.3.4.2 reveals that the appropriate span length falls within the range of 80m to 120m in both cases.

Fig. 7.3.4.2 APPLICABLE SPAN LENGTH COST RATIO DEPENDING ON SPAN LENGTH





7.4 Route Selection

7.4.1 General

The objective of this section is to recommend the most appropriate route for the proposed bridge. Outline structural analysis is prepared for estimating direct construction costs. The costs include both approaches as well as the bridge itself.

7.4.2 Proposed Routes

In the preliminary site investigation, several alternative approach routes for road and railway were selected by using a topographic map drawn to a scale of one-inch to one-mile.

These alternatives were studied in further detail using a map scaled 1:5,000, on which the following natural and manmade constraints obtained from site reconnaissance were plotted:-

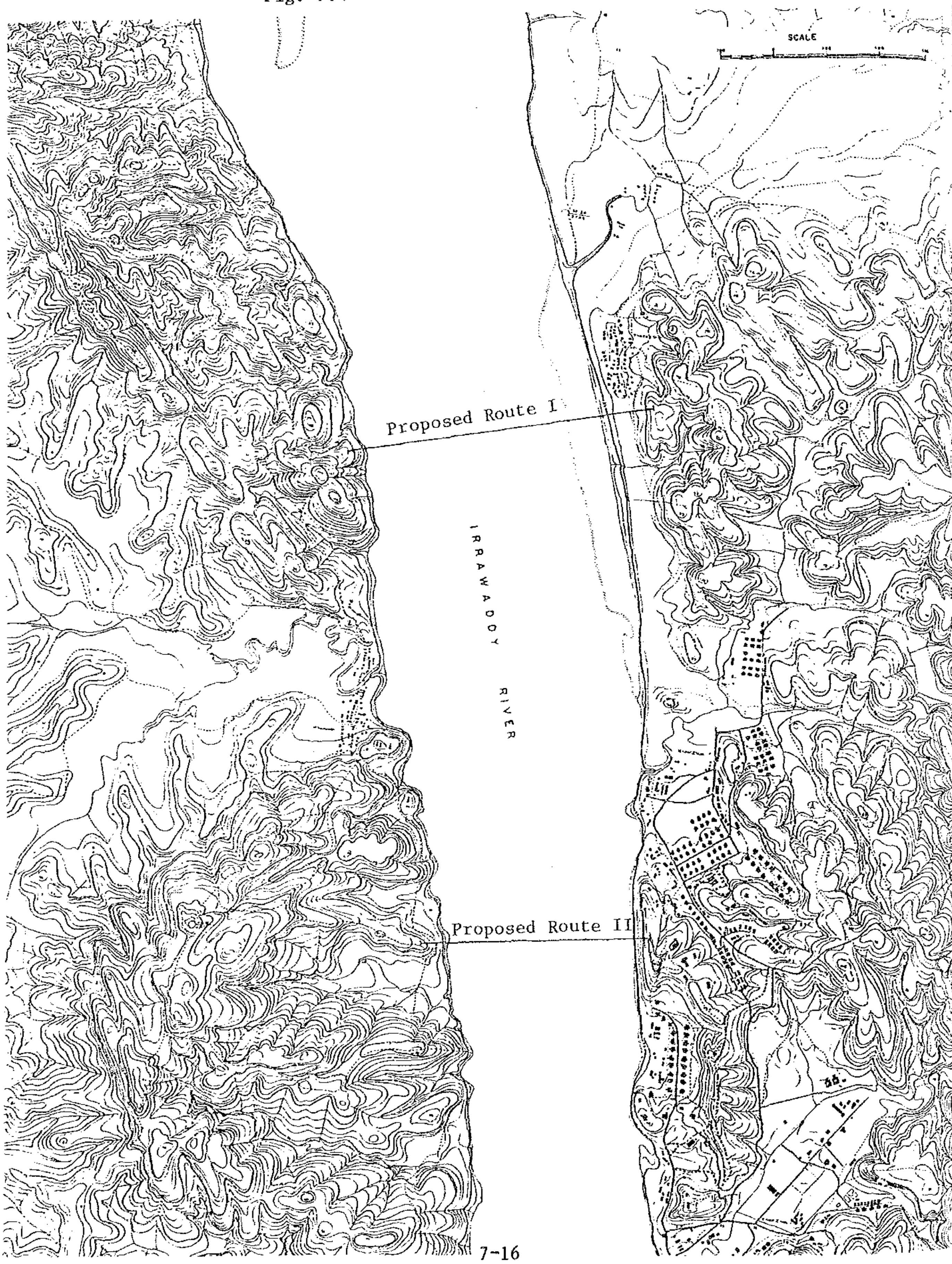
- Natural features: alluvial flat plain, hilly area
- Manmade features: industrial area, village, pagoda

Two proposed routes in Fig. 7.4.2.1 are considered to be the most probable and feasible in the examination area.

The width of waterway at proposed Route I is much wider than that of proposed Route II. However, proposed Route I becomes narrower in the dry season when the water level lowers and some parts of a sand bank on the east side in the river are exposed.

It also should be noted that the Route II passes through a residential area and a fertilizer plant compound on the eastern bank and traverses fairly hilly terrain on the western bank.

Fig. 7.4.2.1 LOCATION OF PROPOSED ROUTES



Major characteristics of the river at the two selected proposed routes are as follows:

Table 7.4.2.1

Items	Route I		Route II	
Approximate river width		1,150.0 m		890.0 m
High water level	E.L.	33.0 m	E.L.	33.0 m
Low water level	E.L.	18.0 m	E.L.	18.0 m
High water depth		27.9 m		26.5 m
Low water depth		12.9 m		11.5 m
Estimated scouring depth		12.0 m		14.0 m
Elevation of bearing stratum	E.L.	26.0 m	E.L.	31.0 m

7.4.3 Bridge Type

As mentioned before, the appropriate structure for the superstructure of the bridge is either PC continuous box girder or steel continuous truss, and the foundation is cast-in-place large diameter pile.

The construction cost of superstructure of steel bridge is higher than that of concrete bridge. However, the substructure of steel bridge cost is less than that of concrete bridge, as the dead load of concrete bridge is higher. Therefore, no significant difference of total construction cost exists between the two.

One representative type of bridge is enough to estimate the construction cost for the route comparison purpose.

The adopted type of bridge for comparison is as follows:

- Traffic function: Simultaneous traffic movement of roadway and railway.
- Configuration of section: Roadway with railway at the side.
- Structural type:
  - superstructure: Steel continuous truss.
  - substructure: Cast-in-place large diameter concrete pile

For the above structure, the outline design is carried out to estimate construction costs of both the alternative routes. The general views of these bridges are shown in Fig. 7.4.3.1 and Fig. 7.4.3.2.

7.4.4 Construction Cost

a) Bridge

The initial cost estimates are shown in Table 7.4.4.1.

Table 7.4.4.1 RELATIVE CONSTRUCTION COSTS FOR THE BRIDGE

(In million Kyat)

Work Item	Route I	Route II
superstructure	294	234
substructure	236	286
Total	530	520

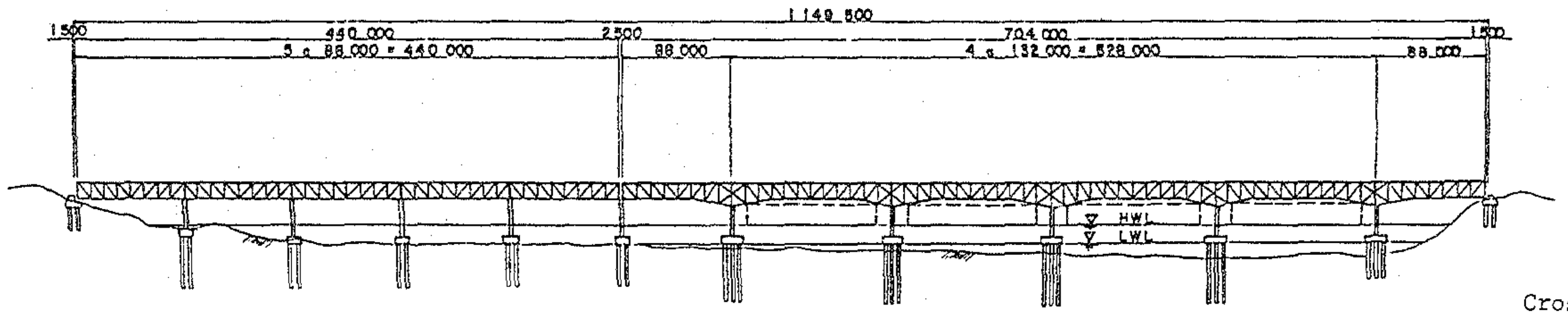
Notes: Exchange rate Kyat 1 = J ¥ 26.

It is notable that the bridge length for proposed Route II corresponds to approximately 80% of that of proposed Route I, while their construction costs are almost identical. This is due to the fact that the river at proposed Route II is consistently deeper than at proposed Route I, requiring the R.C.D pile concrete for proposed Route II to be larger.

b) Approach Road and Railway

Initial construction costs for both road and railway approaches for the 1.6 km section of proposed Routes I & II were estimated from the topographic map prepared to a scale of 1/5,000. Given 55-60 meter elevation of the bridge deck level, the cut and embankment earth work are well balanced, and the construction costs are shown in Table 7.4.4.2.

Fig. 7.4.3.1 ROUTE - I



Cross Section

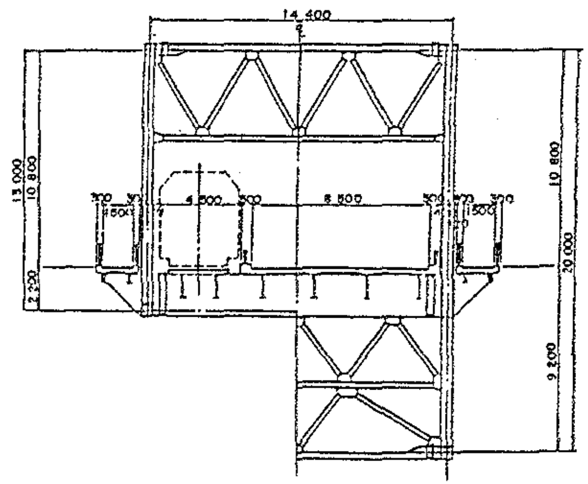


Fig. 7.4.3.2 ROUTE - II

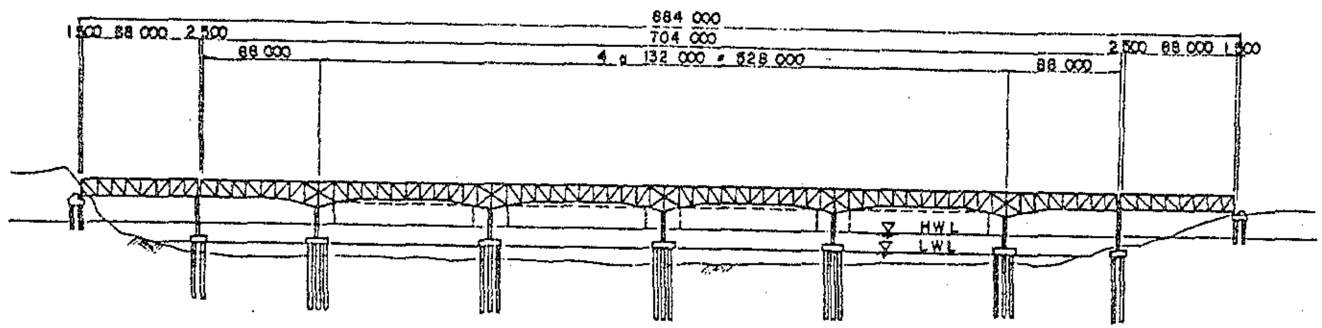


Table 7.4.4.2 RELATIVE CONSTRUCTION COST FOR THE APPROACHES

(In million Kyat)

Work Item	Route No. I	Route No. II
Railway	29	51
Road	10	10
Total	39	61

It is stressed, however, that these costs indicate direct construction costs. The railway construction cost for proposed Route II includes the cost of constructing a tunnel about 600 meters long.

7.4.5 Conclusion

The construction cost comparison for proposed Route I and II is shown in Table 7.4.5.1.

Table 7.4.5.1 RELATIVE CONSTRUCTION COST OF ALTERNATIVES

(In million Kyat)

	Bridge	Approach	Total
Route I	530	39	569
Route II	520	61	581

It is stressed that these estimates do not represent actual construction costs, but are presented herein for comparison purposes only. Engineering fees, custom duties, sales tax, contingencies, etc., are not included in these construction costs. As a result of our comparative study for proposed Route I and II, it is concluded that the construction costs for both routes are similar since the foundation construction cost is a major portion of the bridge cost of the Route II. With respect to the approach, Route II includes an expensive tunnel section. Furthermore, Route II borders a fertilizer plant on the eastern bank and runs through the residential area, which will create difficulties in land acquisition. On balance, we recommend, proposed Route I.

7.5 Selection of Bridge Type

7.5.1 General

In this section, a comparative study of appropriate bridge types for the project is carried out for both the road bridge and the rail-cum-road bridge, based on the conditions discussed before, i.e., the cross sections (Cf. Section 7.2.4), the structural types (Cf. Section 7.3.2 and 7.3.3), the span arrangement (Cf. Section 7.3.4) and the route i.e., Route I (Cf. Section 7.4.5).

In the case of road bridge, the cross section of bridge has no alternative, but for the rail-cum-road bridge, the cross sections divided into three alternative groups depending on traffic function, with comparable construction costs for each.

Therefore, the comparative study of the rail-cum-road bridge is divided into two steps, i.e., the comparison of representative bridge of each group and the comparison of bridges in the group selected.

In this bridge type comparison, prices in March - April 1986 and 1 Kyat = 24 Yen rate are used.

7.5.2 Alternative Bridge Types

Based on the comprehensive study of comparative advantages, the typical alternatives are as follows:

Road Bridge

1. Steel truss bridges
2. PC box girder bridge

Rail-cum-road bridge

A. Simultaneous traffic of roadway and railway
   - A-1 Steel truss bridge (roadway with railway at the side)
   - A-2 PC box girder bridge (roadway with railway at the side)
   - A-3 Steel truss bridge (roadway upper, railway lower)
   - A-4 Combination type of Alt. A-1 and A-2
     (Over the navigation channel: Steel truss bridge)

B. Two-stage construction

B-1 Steel truss bridge (roadway with railway at the side).

C. Alternative traffic between roadway and railway

C-1 Steel truss bridge (roadway with railway at the center).

7.5.3 Road Bridge

The cross sections of both road bridges are shown in Fig. 7.5.3.1 and Fig. 7.5.3.2. The steel weight and concrete volume of them are shown in Table 7.5.3.1.

Table 7.5.3.1 QUANTITIES OF ROAD BRIDGE ALTERNATIVES

Item	Superstructure		Concrete of Substructure		
	Steel	Concrete	Pile	Footing	Pier
Single Deck Truss	6,390ton	-	18,260m^3	12,750m^3	9,350m^3
PC Box Girder	-	12,480m^3	22,730m^3	16,240m^3	5,970m^3

The construction costs for comparison are shown in Table 7.5.3.2. (the cost of temporary work and facility, 43 million kyat is excluded)

Table 7.5.3.2 CONSTRUCTION COST FOR COMPARISON

(In billion yen and million kyat)

Alternative	Superstructure		Substructure		Total	
	Yen	Kyat	Yen	Kyat	Yen	Kyat
Single Deck Truss	5.5	227	5.8	242	11.3	469
PC Box Girder	4.0	167	5.8	241	9.8	408

In the case of road bridge, steel truss type and PC box girder type are studied. Since constraint of deflection limit is less strict for a PC box girder type than for a steel truss bridge, the center hinge structure type is adopted for the PC box girder bridge. It results in less cost than the steel girder bridge.

Fig. 7.5.3.1 STEEL TRUSS

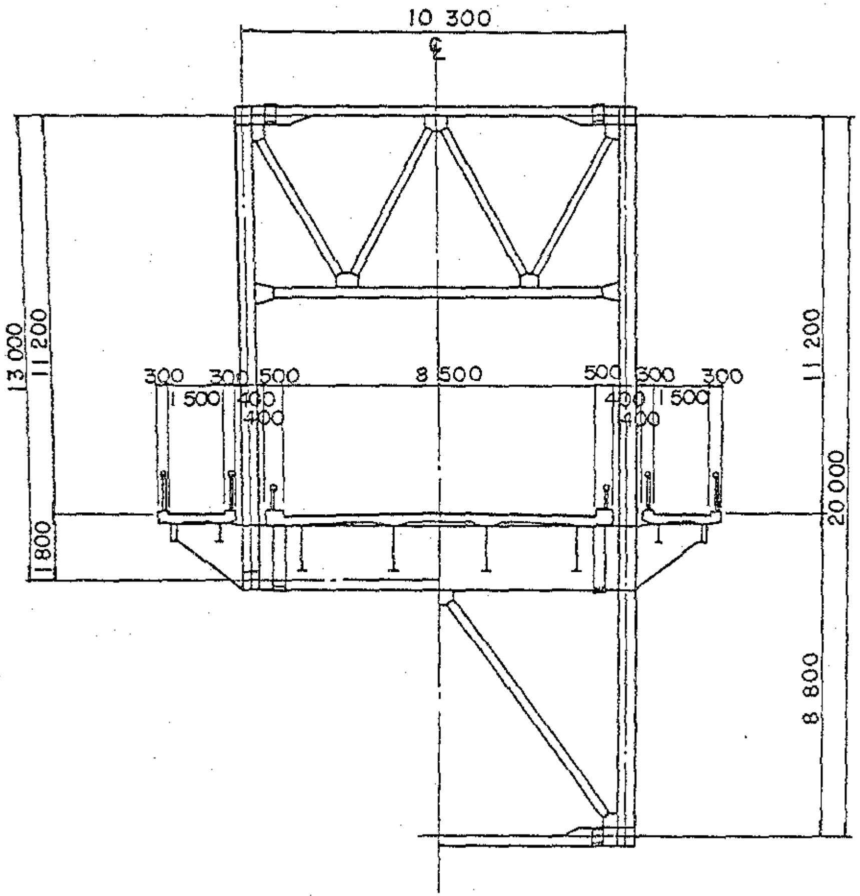
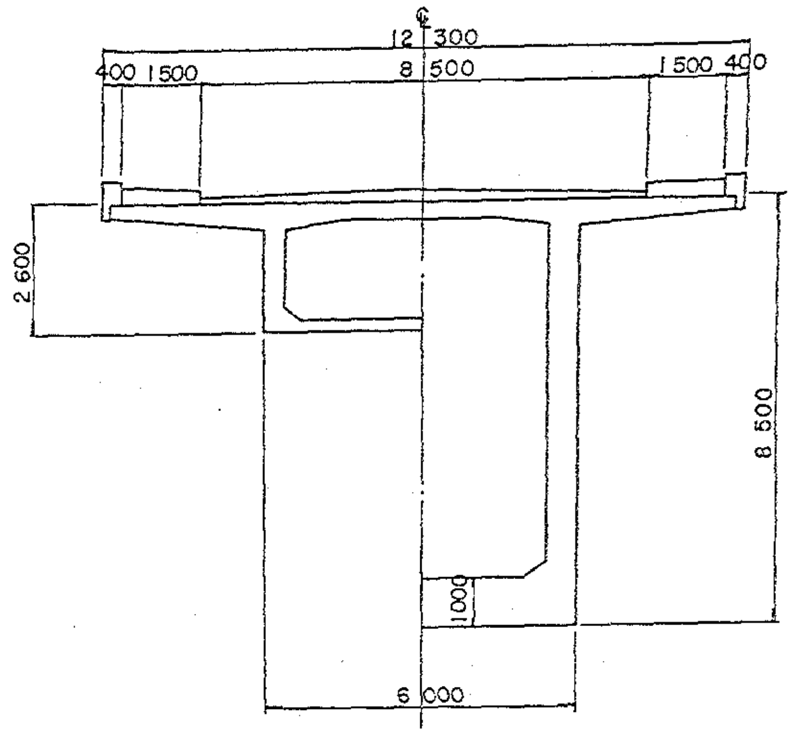


Fig. 7.5.3.2 P.C. BOX GIRDER



From the view point of driver comfort, aesthetic appearance, and traffic smoothness, steel truss bridge is better than PC box bridge. Although construction period of steel truss bridge can be shorter than PC box bridge, it will have a larger maintenance cost. PC box bridge has cost advantages.

7.5.4 Rail-cum-road Bridge

(1) Comparison depending on traffic functions

The aforementioned three traffic alternative are:

A Simultaneous traffic on roadway and railway

B Two-stage construction

C Alternative traffic between road and railway

The cross sections of these alternatives are shown in Fig. 7.5.3.3 to Fig. 7.5.3.7 and the profiles of the bridge in Fig. 7.5.3.8 to Fig. 7.5.3.10. The steel weight and concrete volume of these alternatives are shown in Table 7.5.4.1.

Table 7.5.4.1 QUANTITIES OF RAIL-CUM-ROAD BRIDGE ALTERNATIVES

Item	Superstructure		Concrete of Substructure		
	Steel	Concrete	Pile	Footing	Pier
Single Deck Truss	9,080 ton	-	22,120 m^3	13,250 m^3	10,230 m^3
PC Box Girder	-	19,470 m^3	33,980 m^3	27,350 m^3	10,210 m^3
Double Deck Truss	8,460 ton	-	21,120 m^3	13,250 m^3	8,690 m^3
Roadway - Railway	7,480 ton	-	21,120 m^3	13,250 m^3	9,240 m^3
Truss + PC B. Girder	5,890 ton	7,530 m^3	29,500 m^3	22,900 m^3	12,200 m^3
Two-stage Construction	6,390 4,070	-	21,560 m^3	13,630 m^3	11,660 m^3

For these alternatives, the structural alternative type of steel bridge with the foundation of cast-in-place large diameter concrete piles is adopted in this comparative study as the representative bridge type of each traffic alternative, as mentioned in Section 7.5.2.

ROADWAY WITH RAILWAY AT ONE SIDE

Fig. 7.5.3.3 STEEL TRUSS

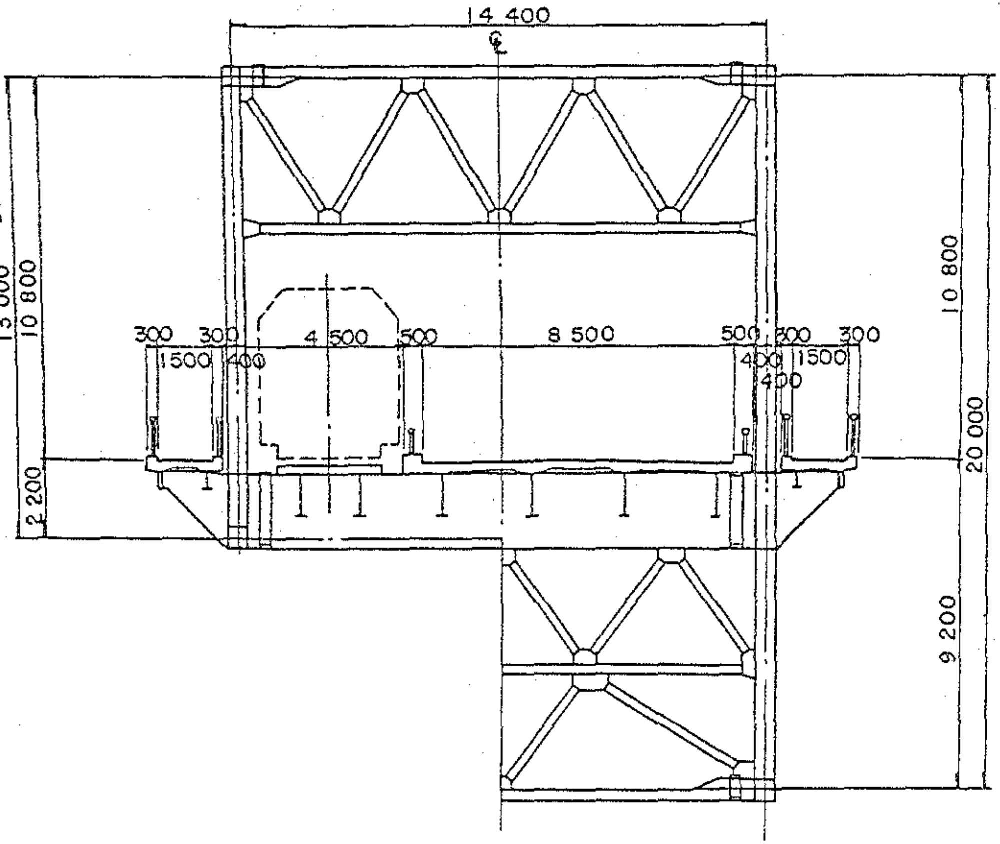


Fig. 7.5.3.4 P.C. BOX GIRDER

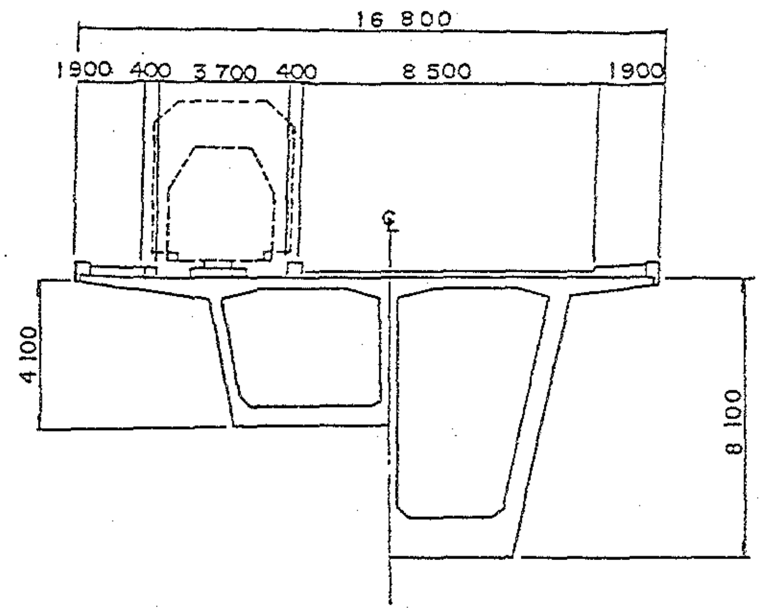


Fig. 7.5.3.5 DOUBLE DECK

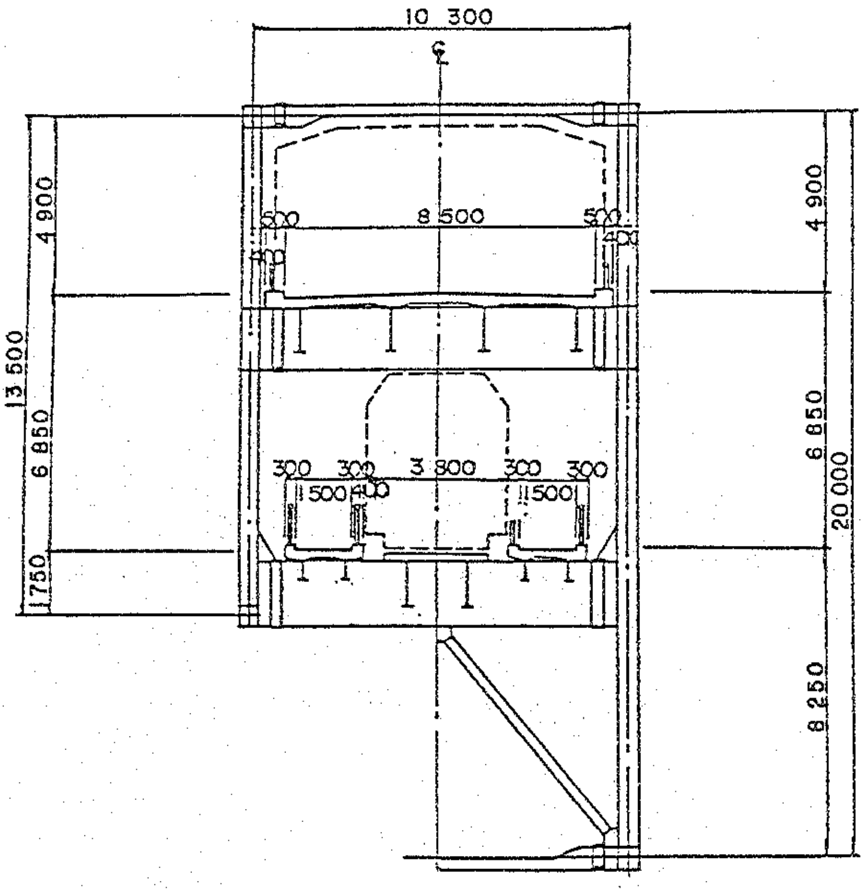


Fig. 7.5.3.6 ROADWAY AND RAILWAY

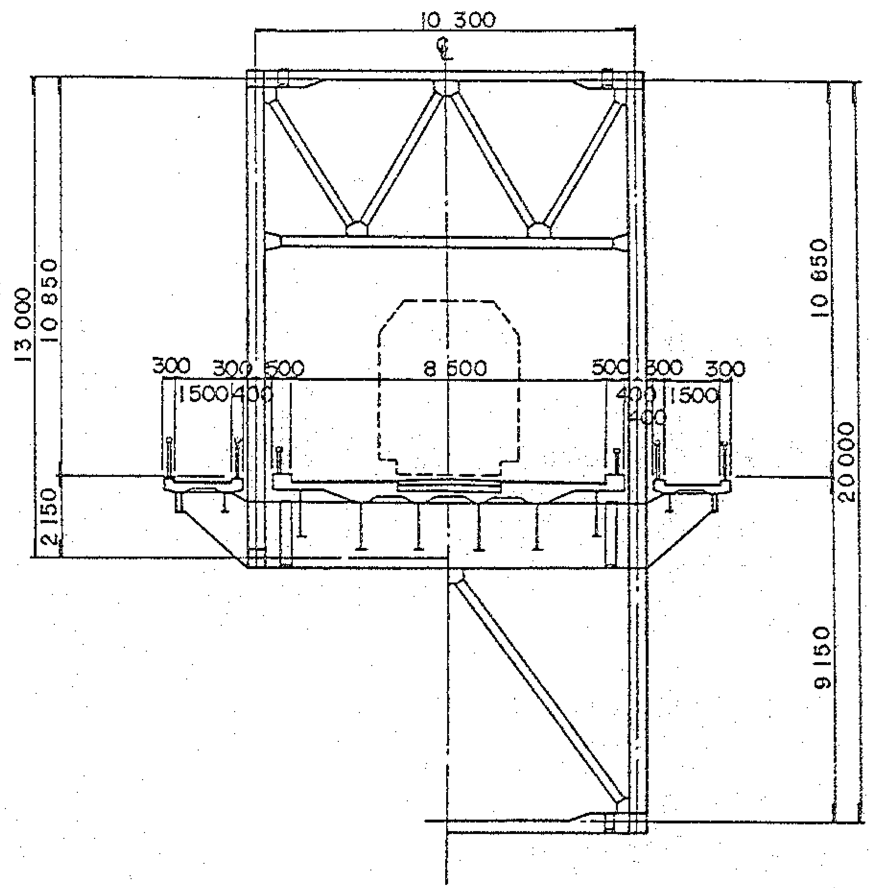
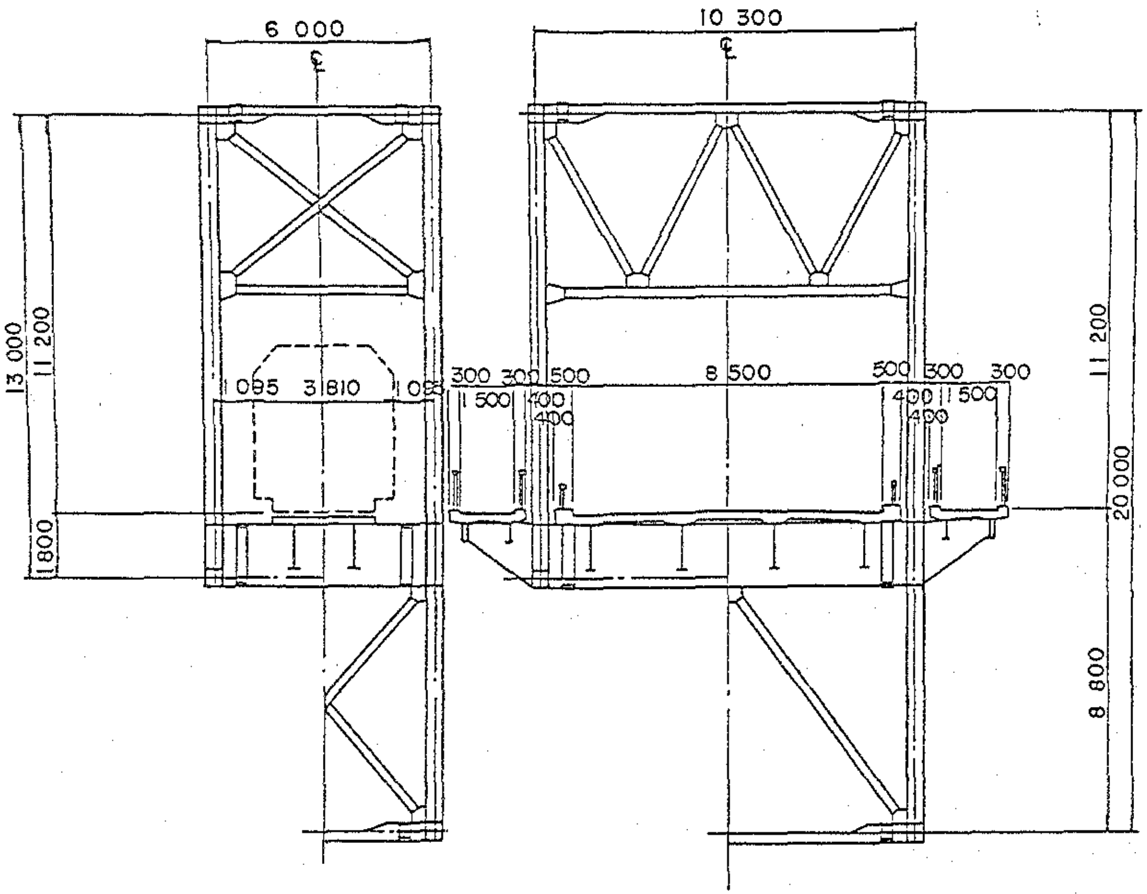


Fig. 7.5.3.7 STAGE-CONSTRUCTION



NOTE : RAILWAY FIRST
ROADWAY LATER

Fig. 7.5.3.8 PROFILE OF STEEL TRUSS BRIDGE

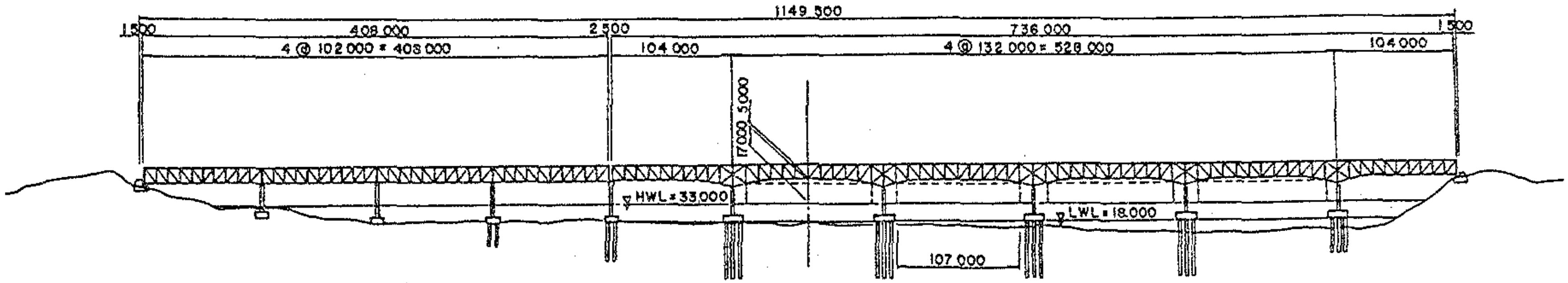


Fig. 7.5.3.9 PROFILE OF P.C. BOX GIRDER BRIDGE

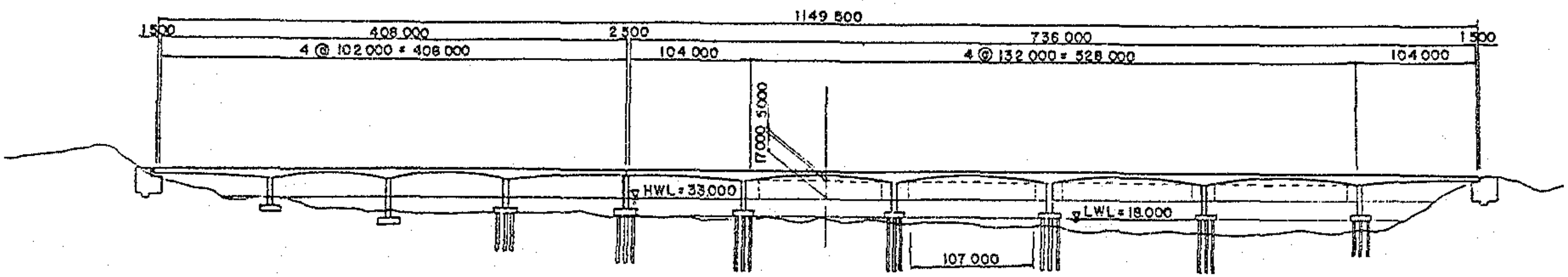
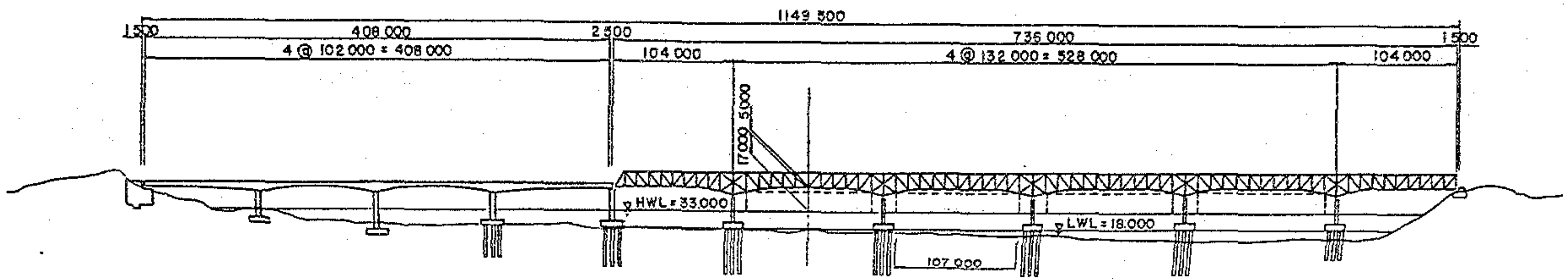


Fig. 7.5.3.10 PROFILE OF TRUSS + P.C. BRIDGE



The construction costs for comparison are shown in Table 7.5.4.2. (the cost of temporary work and facility, 43 million kyat, is excluded)

Table 7.5.4.2 CONSTRUCTION COSTS FOR COMPARISON

(In billion yen and million kyat)

Alternative		Superstructure		Substructure		Total	
		Yen	Kyat	Yen	Kyat	Yen	Kyat
A	A-1: Single Deck Truss	6.8	283	6.1	254	12.9	537
A	A-2: PC Box Girder	5.1	213	7.7	321	12.8	534
A	A-3: Double Deck Truss	6.6	273	6.1	254	12.7	527
A	A-4: Truss + PC B. Girder	6.4	266	6.7	279	131.1545	
B	Two-Stage Constructions	5.5 (+ 4.0)	227 (+ 167))	6.2	258	11.7 (+ 4.0)	485 (+ 16)
C	Road/Railway	6.1	252	6.1	254	12.2	501

The alternative C has the lowest construction cost, but the least traffic serviceability.

The alternative B is a plan to construct in two stages. Although the first stage cost is the lowest among these types, it requires a substantial cost in the second stage resulting in the highest cost among them.

The construction cost of alternative A is in the middle between the two alternatives, and the alternative would have the best traffic serviceability provided the approaches of railway and roadway to the bridge could be completed at the same time.

From the discussions above, the study team recommends alternative A, principally because of traffic serviceability, as it is the most important factor of bridge usage.

(2) Comparison of bridge types among the Group A

The construction costs of alternative bridge types in the traffic function A are shown in Table 7.5.4.2.

The construction cost of alternative A-3 needs to accompany the construction cost of its approach structures, as only this alternative needs approach structures discussed in Section 7.6. The construction cost of the approach structures is shown in Table 7.6.4.1.

In this group, the construction costs are almost the same. When maintenance cost is considered, its cost in the case of concrete bridge is generally lower than that of steel bridge.

Both the truss bridge and PC box girder bridge would adopt cantilever erections which could construct the superstructure over the river without any scaffolding. However, a truss bridge is easier to erect than PC box girder bridges, because the construction of truss bridge needs only one-third the cantilevering erection sets, assuming that the erection work starts and finishes at the same time. When comparing alternative A-1 and A-3, the latter would be rather easier to construct because of symetrical cross sections.

Regarding appearance, three alternatives from A-1 to A-3 are almost the same. However the alternative A-4 is inferior to the other three, because of change of bridge width and height at the joint between the truss and the box girder.

As for the serviceability of traffic, alternative A-4 is not superior to the other three alternative as the change of the bridges width would impede pedestrian traffic. Among the other three alternative, alternative A-2 has good serviceability for the road traffic, because the cars do not have to run through the truss. Although alternative A-3 has an advantage because both roadway and railway traffic are completely separated and a passing train on the bridge has no effect on car drivers on the bridge. Guardmen and maintenance service cost takes more two times than others.

7.5.5 Selection

1. Road bridge

   PC box girder type was selected for the road bridge after studies in engineering, cost and economic analysis.

2. Rail-cum-road bridge

   With the similar comparative studies, the following three types were identified in the priority group. However, selection of one type among the group were made after the interim discussion hold in August 1986.

   A-1 Steel single deck truss type

   A-2 PC box girder type

   A-3 Steel double deck truss type

   Construction Corporation and the Study Team agreed to select A-1 for the works in preliminary and cost refinement in Phase II studies.

7.6 Approach Road and Railway

7.6.1 General

The feasibility study covers only the 1-mile section of approaches 60th sides of the bridge. However, the approach road and railway plans which are to be implemented by Government of Burma in advance to the completion of the bridge project are identified as in this section.

The alignments of the approach road and railway were studied on the topographic map at a scale of 1:5,000 taking into consideration of findings of the field survey. The preliminary design of the approach road and railway was carried out to meet the basic standard and requirement as mentioned in the subsection 7.2 of the chapter. They are shown in Drawing 26 and 27.

The three types of the approach road and railway for the both road bridge and rail-cum-road bridge were studied.

7.6.2 Approach Road

The proposed route starts at the 194/5 mℓ point on Rangoon-Prome-Mandalay road and runs along the sides of hills. Upto the midway the route uses the existing fertilizer road, and then divert to the northward direction. Thus the approach road avoids the MOE areas where fertilizer plant, residential area and relevant facilities have been built on the eastern bank. In the western bank the route starts at the 24/6 mℓ point on Western Highway from Okshitpin junction and passes along the sides of hills. To construct this stretch it is possible to contribute to access to social services for local people living along the river side. The total length of the approach roads is appropimately 27 miles (east side 7 mℓ and westside 20 mℓ) excluding bridge stretch. The construction cost is estimated at 54 million kyats, and the construction period is seven years upto 1992 at the latest.

In the case of the double deck type, the crossing of the railway and the road will be constructed at the both end of the bridge. No technical problems will be found, though it will require additional cost for structure.

The Location of the approach road is shown in Fig. 7.6.2.1.

7.6.3 Approach Railway

7.6.3.1 Location

It is planned by BRC to construct the new Prome-Kyawzwa railway line and the new Kyangin-Myawaddy railway line as mentioned in Paragraph 3.2.3.4. These two railway lines will be connected to each other by the proposed Irrawaddy bridge. The conceptual alignment of those railway lines planned by BRC is shown in Fig. 7.6.3.1. The survey work of the new Prome-Kyawzwa railway line started in March 1986, while for a length of 40 miles from Kyangin to Natmauk on the new Kyangin-Myawaddy railway line ground survey work has already been completed. The construction cost of the lines are estimated by BRC at K 47.5 million for the Prome-Kyawzwa line and K 262.5 million for the Kyangin-Myawaddy line. The funds will be furnished from the general government budget. The railways will be completed in 1992/93.

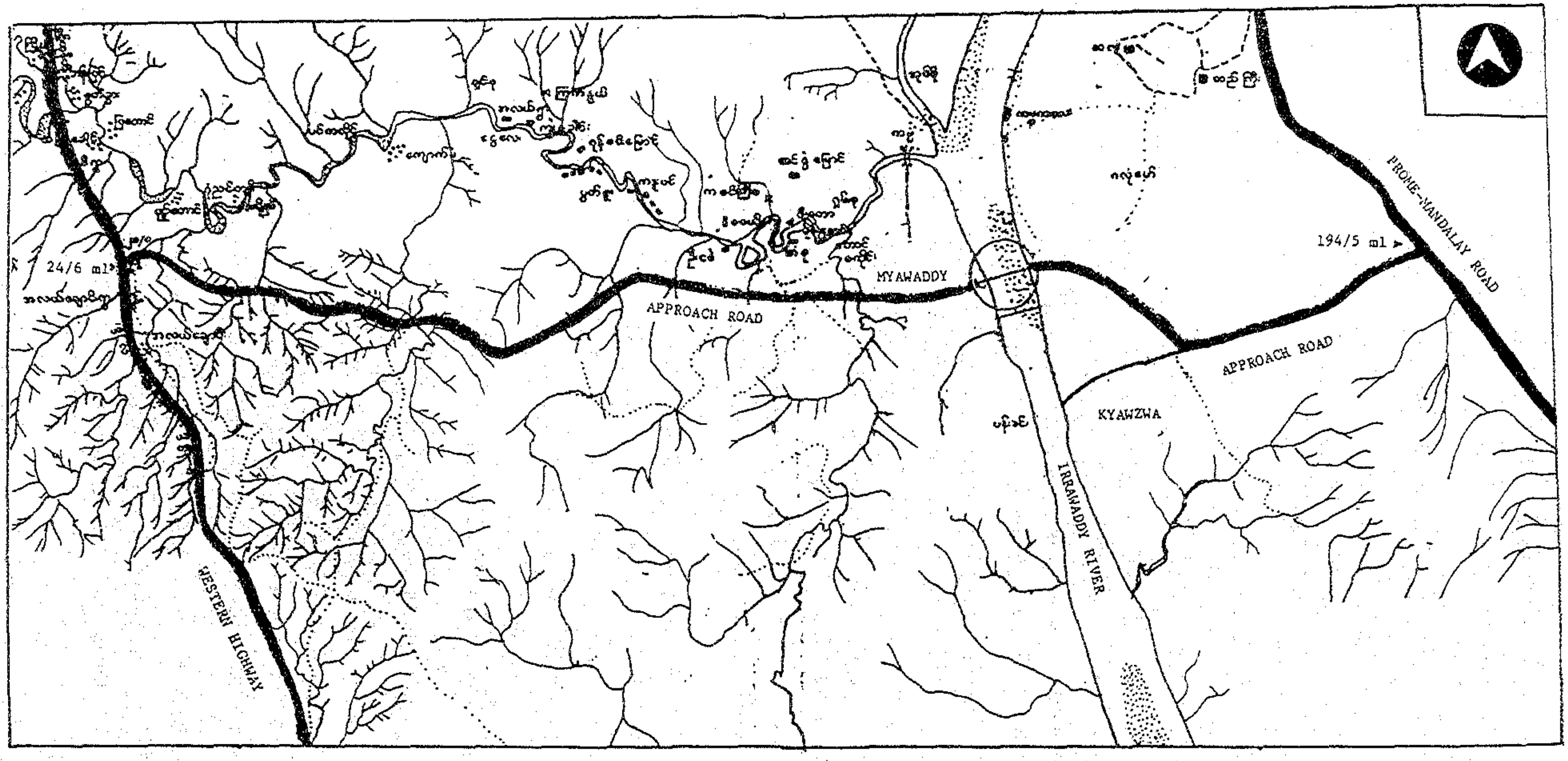
The topography of the construction site of the new line is rather hilly on the east side of the river and rather mountainous on the west side. However, no tunnels would be constructed on both sides. The location of the approach railway is shown in Fig. 7.6.3.1.

7.6.3.2 Operation System

1) Train Route

The trains to be operated on the proposed bridge will run on the Prome Line and the Western Line. The extension railways between Prome and the bridge, and between Kyangin and the bridge will be completed simultaneously to form a Rangoon-Prome-Kyawzwa-Myawaddy-Kyangin-Henzada-Bassein railway network. Passenger and freight trains will be operated on the length of the entire section or limited sections in response to traffic demand.

Fig. 7.6.2.1 PROPOSED APPROACH ROAD ALIGNMENT



2) Kind of Train

Passenger, freight and/or mixed trains will be operated by diesel locomotives. The recommended horse power of the diesel locomotives is in excess of 1,500 HP due to steep approach gradients.

3) Number of Trains

The number of trains will be estimated once the future traffic demand is fixed. According to a preliminary analysis, two or three trains including passenger, freight and/or mixed train on each direction will be operated when these lines are connected. The number of trains is not expected to increase sharply because of the slow increase in future traffic volumes.

4) Train Speed

The maximum speed of trains will be 35 miles per hour (56 km/h) for passenger trains and 25 miles per hour (40 km/h) for freight trains as stipulated in the current BRC operating rules. However, BRC will likely raise train speed to 60 miles per hour (nearly 100 km/h).

5) Block and Signalling System

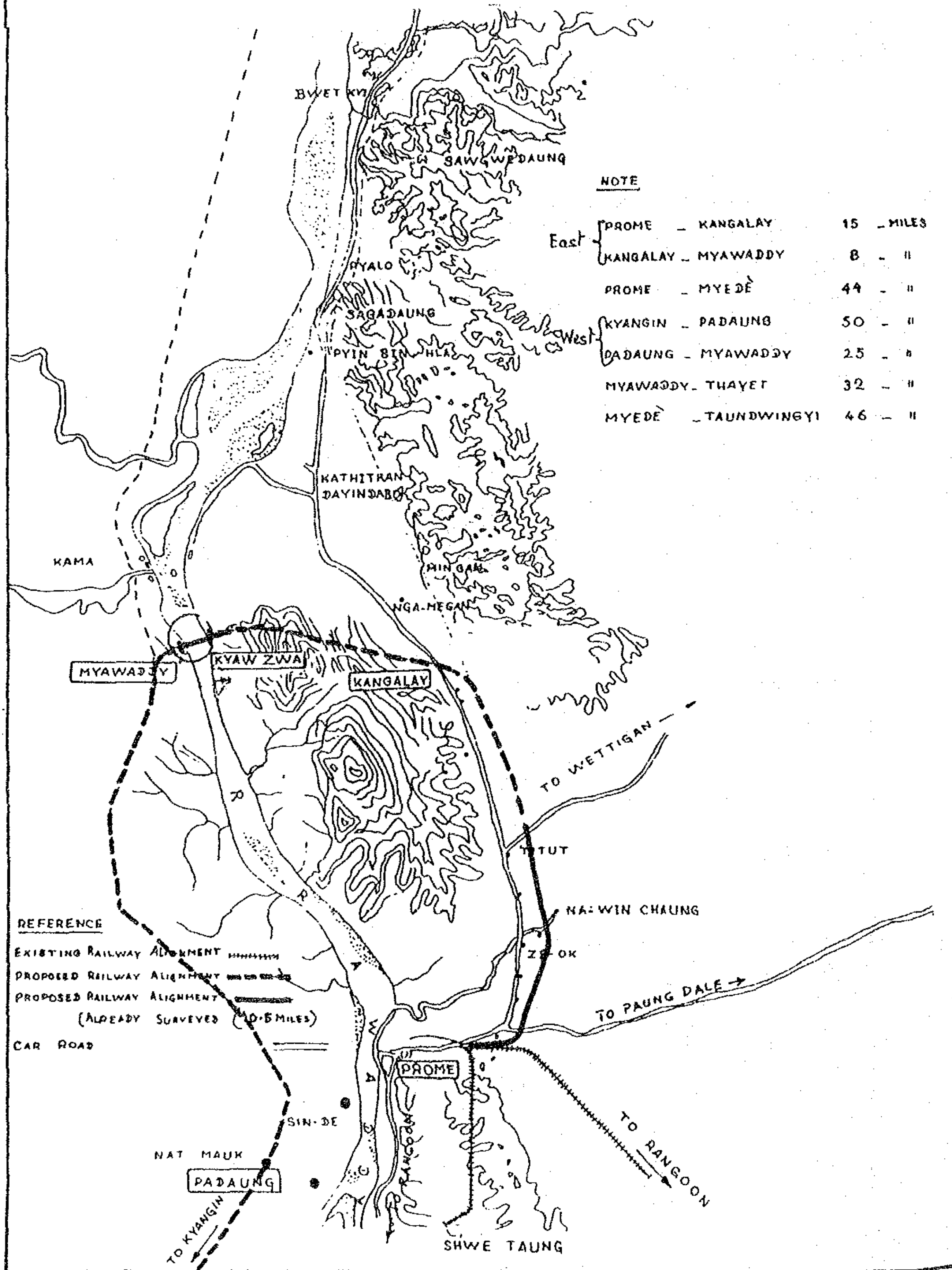
Because the number of train is small, the number of tracks on the proposed bridge and the approach sections will be single, and no crossing stations will be constructed within one mile from the end of the bridge on both side.

On the existing Prome Line the token block system is used, while on the existing Western Line the block system is supported only by Morse Telegraph. The block system to be used on the proposed bridge and the extension sections will be the same token block system used on the existing Prome Line. No signals are necessary on the bridge or within one mile of it.

PROME_MYEDE_TAUNGDWINGYI AND KYANGIN_THAYET

Fig. 7.6.3.1 PROPOSED RAILWAY ALIGNMENT

SCALE 1 INCH = 4 MILES



CHAPTER 8 PRELIMINARY DESIGN

CHAPTER 8 PRELIMINARY DESIGN

8.1 General

Based upon the results of CHAPTER 7 ENGINEERING STUDY where the bridge location, cross section of roadway and railway, span arrangement, bridge type and the adequate design standards were fixed, preliminary design has been carried out for the both types of bridge, i.e. the roadbridge and railway-cum-road bridge.

This chapter describes detailed design criteria, principle of structural analysis and construction plan assumed in the preliminary design work.

8.2 Design Criteria

8.2.1 General

The concrete standards and conditions will be settled in these paragraphs, based on the basic conditions stipulated in Chapter 7.

8.2.2 Clearance and Design Standards

The required clearances for traffic such as vessels navigating under the bridge, and the design standards for the road and railway are as shown in Section 7.2.2.

8.2.3 Loads

Types of Load considered:

o Main-Load (P)	(1)	Dead Load (D)
	(2)	Roadway Live Load (LV)
	(3)	Railway Live Load (LT)
	(4)	Pedestrian Load (LC)
	(5)	Impact Load (I)
o Sub-Load (S)	(6)	Train's Vibrational effect (LF)
	(7)	Train's Traction and Braking Force (B)
	(8)	Wind Pressure (W)
	(9)	Effect of Temperature (T)
	(10)	Effect of Earthquake (E)
o Special Load (PA)	(11)	Effect of Settlement (GD)
	(12)	Temporary Load During Construction (ER)
	(13)	Collision Force (CO)

Details of each load type are as follows:-

(1) Dead Load (D)

To comply with JSHB.

(2) Roadway Live Load (LV)

TL-20, specified in JSHB, together with a 60 ton trailer load (Cf. Section 7.2.3) are adopted.

(3) Railway Live Load (LT)

As specified in Section 7.2.3.

(4) Pedestrian Load (LC)

500 Kg/m^2. Details of load conditions comply with JSHB.

(5) Impact Load (I)

Impact of roadway and railway traffic comply with JSHB and SDJR, respectively.

In the case of a steel truss rail-cum-road bridge, both roadway and railway impact factors are adopted for the respective loads, as there are direct effects from both of these live loads.

As the design speed of the railway is less than that in Japan, the reduced factor i in following formula is adopted:-

$$i = \frac{0.48}{L^{0.2}} + \frac{10}{65 + L}, \quad \text{where } L = \text{span length}$$

(6) Train's Vibrational Effect

The forces shown in Fig. 8.2.3.1 are adopted. The details conform to SDJNR.

(7) Train Traction and Braking Force (B)

The longitudinal load when the train starts and stops is 25% of the driving axle load and 15% of the train load, respectively.

(8) Wind Pressure (W)

The wind pressure is assumed to be 300 Kg/m^2. Details comply with JSHB. For the train, force due to wind pressure is calculated as shown in Fig. 8.2.3.2.

(9) Effects of Temperature (T)

The temperature change ranges from 5°C to 55°C, and the mean temperature is assumed to be between 25°C and 30°C depending on the site condition.

(10) Effect of Earthquake (E)

As specified in Section 7.2.3, where K_h=0.125

(11) Effect of Settlement (GD)

No effect is considered.

(12) Temporary Load (ER) during construction

The construction loads which affect the bridge structure are considered.

(13) Collision Force

$P = 0.1 * w * v$

where, P : Collision Force (t)

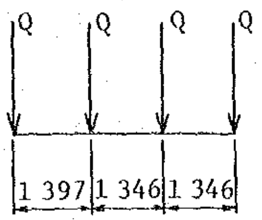
W : Weight of Drifting Item (t)

v : Surfacial Speed of Water (m/sec)

The heaviest ship in Inland Water Transport Corporation is 2,000 tons.

Maximum surface speed of water was surveyed 3.6 m/sec at Kyawzawa.

Fig. 8.2.3.1 TRAIN'S VIBRATION EFFECTS

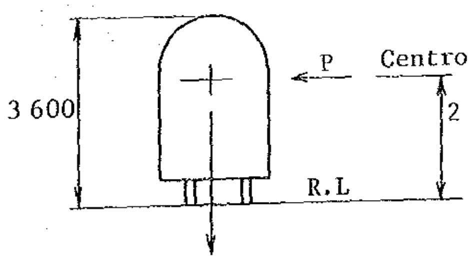


Q = Horizontal force

= 15% of axle load

= 13.2 x 0.15 = 1.98 t

Fig. 8.2.3.2 WIND PRESSURE TO TRAIN



P: Resultant force of wind pressure

1 800 kg/m (Trains weight shall be 1.8 k/m when considering wind effects)

8.2.4 Material

Specification of main materials:

(1) Concrete	o PC-Box Girder	:	σck = 350 kg/cm^2
	o Deck slab	:	σck = 240 Kg/cm^2
	o Substructure	:	σck = 240 Kg/cm^2
	o RCD Pile	:	σck = 350 Kg/cm^2

Note: σck = specified compressive strength at 28 days.

(2) Reinforcing bars : SD30

(3) PC Tendons : SBPR 95/120 ϕ32

(4) Steel truss members : SS41, SM50Y, SM58, etc.

8.2.5 Allowable Stresses

Allowable stresses of the main materials, to which factors are applied in accordance with the table 8.2.5.1, are as follows:-

(1) Concrete

The allowable compressive stresses

o PC-Box Girder	:	σca = 125 kg/cm^2
o Deck slab	:	σca = 80 Kg/cm^2
o Substructure	:	σca = 80 Kg/cm^2
o RCD Pile	:	σca = 80 Kg/cm^2

The details comply with JSHB.

(2) Reinforcing Bars

The allowable tensile stress is 1800 Kg/cm^2. The details comply with JSHB.

Table 8.2.5.1 FACTORS FOR ALLOWABLE STRESSES

	Main - Load (P)					Sub - Load (S)					S-Load (PA)	Factor
	D	L_V	L_T	L_C	I	L_F	B	W	T	E	E_R	(α)
1	○	○	○	○	○							1.00*
2	○	○	○	○	○				○			1.15
3	○	○	○	○	○			○				1.25
4	○	○	○	○	○			○	○			1.35
5	○	○	○	○	○		○					1.25
6	○							○				1.20
7	○		○		○					○		1.70
8	○		○		○				○	○		1.70
9											○	1.25
10	○	○	○	○	○	○						1.25
11	○	○	○	○	○	○		○				1.35
12	○	○	○	○	○		○	○				1.35
13						○		○				1.25
14							○	○				1.25

Note: α = Factors for allowable stresses

* In case of Steel Truss the factor of 1.25 is to be adopted in following load combinations:

o Main truss member : $(D) + (L_V) + (L_T)$

o Cross beam : $(D) + (L_V) + (L_T) + (TT)$
— a truck and a trailer are to be adopted

o Stringer : $(D) + (TT)$

where TT: 60 ton trailer load.

(3) PC Tendons

The allowable tensile stress is 71.25 Kg/mm^2. The details of allowable stress for various groups of loading and combinations of sectional forces comply with JSHB.

(4) Steel Truss Members

The allowable stress specified by JSHB and SDJNR design standards are as follows:

SPECIFICATION	SS41	SM50Y	SM58
JSHB	1400	2100	2600
SDJNR	1500	2200	2600

Note: Unit=Kg/cm^2

In accordance with each bridge member that carries roadway loading or railway loading, the design standard is defined as follows:

a) Roadway load supporting member : JSHB
b) Railway load supporting member : SDJNR
c) Combination load supporting member : JSHB

The details of allowable stress for various groups of loading and/or combinations of sectional forces comply with their respective design standards.

8.2.6 Fatigue

(1) PC Box Girder

No effect is considered.

(2) Steel truss

In the case of a road bridge it is not necessary to consider fatigue because no significant change of stress due to live load occurs.

However, in case of a railway bridge the effect cannot be neglected because of large amplitude of stress resulting from the live load.

Therefore, in this design the fatigue effect due to the railway live loading is considered.

The details of analysis comply with SDJNR.

8.2.7 Maximum deflection due to live loading

* L denotes span length in meter in the following formulae.

* x denotes maximum deflection

(1) PC Box girder : $x = L/600$

(2) Steel Truss Bridge

The deflections are limited to the following values, depending on the type of truss member.

- Main Truss Chord Member

  As it seems highly unlikely that the railway and roadway will be simultaneously subjected to full live load so as to produce maximum deflection in the main truss, the limits of this deflection are set as below:-

  - For roadway maximum live load : $x = L/600$
  - For railway maximum live load : $x = L/1000$

- Cross Beam

  As there is a high possibility that this member is simultaneously influenced by both live loads, the limits of deflection, for the portions effected by these loads, are as follows:-

  - For beams supporting the roadway : $x = L/(20,000/L)$
  - For beams supporting the railway : x = 5 mm at intermediate cross beams (as measured at the stringers) and 4 mm at end beams

· Stringer

As the stringer is never influenced by combined loading, the limits of deflection are set as follows:-

- For roadway stringer : x = L/(20,000/L)
- For railway stringer : at end span x = L/2000

8.2.8 Stability of Substructure

The stability of substructures complies with JSHB, with satisfaction of the following conditions.

(1) Spread Foundation

- o The bearing reaction is less than the allowable values specified in JSHB.
- o Eccentricity of the resultant vertical forces vector (due to basic loading conditions) is fall within the middle one third of the basement. The resultant forces vector considering seismic loading conditions, shall falls within the middle two thirds of basement.
- o Sliding safety factor : 1.5 for basic loading conditions and
  1.2 for earthquake conditions

(2) Pile Bent Foundation

- o The pile reaction is less than the allowable values specified in JSHB.
- o The allowable displacement of a pile is considered to be less than 1.0 cm and 1.5 cm at the river bed surface, in case of basic loading and earthquake respectively.

8.2.9 Scouring

Scouring depths of 12 m to 14 m, studied in Phase I were applied to the preliminary design in Phase II, while the depths analyzed in Phase II by the site investigation carried out when the river reached the high water level are expected to be 10 m to 12 m. Although the design has been checked with the new data, it turned out that the difference in scouring depth mentioned above doesn't produce so significant effect that the design should be revised. (cf. Appendix Table 8.2.9) Further study will be done in the detailed design using the new datum.

8.3 Structural Analysis

8.3.1 General

The structural types of superstructures in the two bridges proposed are different that is PC Box Girder for the road bridge and Steel Truss for the rail-cum-road bridge; while the structural types of substructures are the same except the configuration of the piers for both types.

8.3.2 Superstructure

(1) PC Box Girder Bridge - Road Bridge

The whole bridge structure consists of multi T-shaped frames. Each T-frame comprizes a column and two cantilevered arms (PC Box Girder). The end of each arm is joined to the adjacent T-frame by a horizontally sliding hinge.

The PC box girders (arm of T-frame) is to be constructed segment by segment by means of in-situ concrete cantilever erection from pier top toward span center. The characteristics of this structural type and construction method are as follows. As the bridge structure itself is able to support whole the erection force, no special support is required during construction. The magnitude and the distribution of the bending moment occuring during construction are almost the same as that due to the design load, and no special reinforcement of the girder for the erection is required.

Another merit of this construction method is that the repetition of the work soon promotes expertize in the labour force.

Spans 102 m and 132 m are alloted to the sections over the shallows and over the navigational channels respectively.

The points of analysis are as follows:-

- Dimension of girder section

The width of the upper slab is determined to meet the requirements of the configuration of the road cross section, its thickness is governed by the traffic load intensity and the haunched portion is required to accommodate the longitudinal PC tendons.

The girder depth and width of box sections are fixed based upon extensive experience, and the thickness of web and lower slab are derived from the results of structural analysis.

Although the web is a member of the box section which resists transversal bending moment, it also resists shearing stress caused in the main girder and accommodate longitudinal and diagonal PC tendons.

The web thickness is determined so as both to accommodate the necessary number of tendons and to cover diagonal tensile stress due to the shearing force. The diagonal PC tendons directly work to reduce the diagonal tensile stress transfering compression to the web concrete.

The lower slab resists compressive stress due to longitudinal negative moment, and the thickness is governed by same. However, in the end span, a longitudinal positive moment occurs, and longitudinal PC tendons are required in the lower slab.

These dimensions and the number of PC tendons are adjusted by trial-and-error to get the optimum solution.

- Structural Analysis

The structural analysis is carried out so as to obtain safety against both design load and temporary loading during construction.

Although the analysis is carried out, from practical point of view, independently between longitudinal main frame and transversal girder section, these analyses correlate to each other.

The results of the analysis are shown in Appendix Table 8.3.2.1 -8.3.2.6.

(2) Truss Bridge - Rail-Cum-Road-Bridge

The bridge superstructure consists of two continuous trusses. One of them comprises 6 spans (104+4x132+104) over the navigational channels, and the other 4 spans (4x104) over the shallows. The roadway and railway are accommodated inside the truss members in parallel, and the separated two sidewalks are placed outside of the main truss chord members.

The superstructure will be constructed without using any scaffold in the river so that construction may proceed even in the rainy season. Three construction methods, that is, large segment erection, incrementally launching and cantilever erection, will meet this requirement. Among these methods the cantilever erection is the most likely suitable for this construction site. The temporary tower, from which the temporary cables suspend the truss member during the erection, is considered to be adopted in order that the large sectional forces caused during construction are not govern the dimension of truss member section. Without this temporary tower the quantity of truss steel would be increased by approximately 10%.

The bridge structure has to resist significant torsional force due to imbalanced dead and live loads. As a countermeasure, stiff intermediate cross beams, which support stringers, are used together with intermediate sway bracings between the upper chord members.

Fixed bearings are set on the abutments on both banks. On all the piers longitudinally sliding bearings are arranged. This concept permits significant economizing in the design for the high piers and deep pile foundations reducing the longitudinal force caused by earthquakes. As only one expansion joint is prepared at the joint of two separated continuous tress structures, a substantial amount of expansion and contraction is center at the joint (cf Appendix Fig. 8.3.2.1).

- Main Dimension

The width is determined from the widths of the roadway and railway. However, the height is determined from many factors, such as span length, kind of loads, quality of truss steel, restriction of deflection, etc. In this design, the varying truss height is selected by trial-and-error analysis, so that the truss members have adequate sections. As for the configuration of the height, the same heights of main spans at the center are selected in the approach spans, which does not vary, for aesthetic reasons.

- Analysis

To get an economical solution, the member sections are selected from three alternative sections consisting of different, steel material qualities (SS41, SM50Y, SM58). When the qualities in a member are different the section are chosen considering smoothness in mechanical properties as a whole. In the analysis conventional methods are adopted.

Using 0-1 method, the loads between the main two truss chord members are distributed to the members which support the loads. The truss chord members are analyzed in a plain frame, assuming that all the members are joined by hinges. The flooring systems are derived from simple formulas specified in JSHB and JOJNR. However, the intermediate sway bracers under the upper chord for the torsion cannot be solved using this conventional method, and are thus experimentally determined. In detail design this amount will be checked using space truss frame analysis.

The results are shown in Appendix Fig. 8.3.2.2 and Appendix Table 8.3.2.7 to 8.3.2.14.

8.3.3 Substructure

(1) Pier

- Road Bridge

As the column of the pier rotates and deforms as a result of the change of cantilever erection load in T-frame when it is not joined to adjacent girders, the column must have enough stiffness to ensure adequate girder level in order to keep the smooth progress of the construction work. In this design the pier has a box section to make it stiff.

The width of the column is the same to that of PC box girder, and the thickness of the box section is determined so as to bear the load acting upon it.

- Rail-cum-Road Bridge

As the distance between bearing shoes, which support the truss outside the rail-cum-road way is wide, the pier also will be wide enough to accomodate both bearings. In this design, two brackets are prepared for the side of the pier column top to reduce the pier weight by minimizing the width of the column.

The thickness of column is governed by the space required for putting the bearing shoes on it, and its width is determined so as to complement the stress caused by the loading in case of earthquake. Common dimensions of 7 meters wide and 3 meters thick are adopted throughout the bridge in order to simplify the construction procedure. However, this size is not possible to be adopted for pier P_5 because two bearings of two continuous trusses put in the longitudinal direction at this point, and thus pier dimensions of 7 meters wide and 5 meters thick are adopted.

A box section is adopted only for P_5 pier which has especially large section in order to reduce its weight, while the section without void is adopted for other piers in order to simplify the construction work.

The width and cantilever length of the brackets are determined to obtain the space required by the main truss chord members and width of column and its depth is governed by the reaction force of bearing shoes. However, common dimensions are also adopted for each bracket.

(2) Foundation

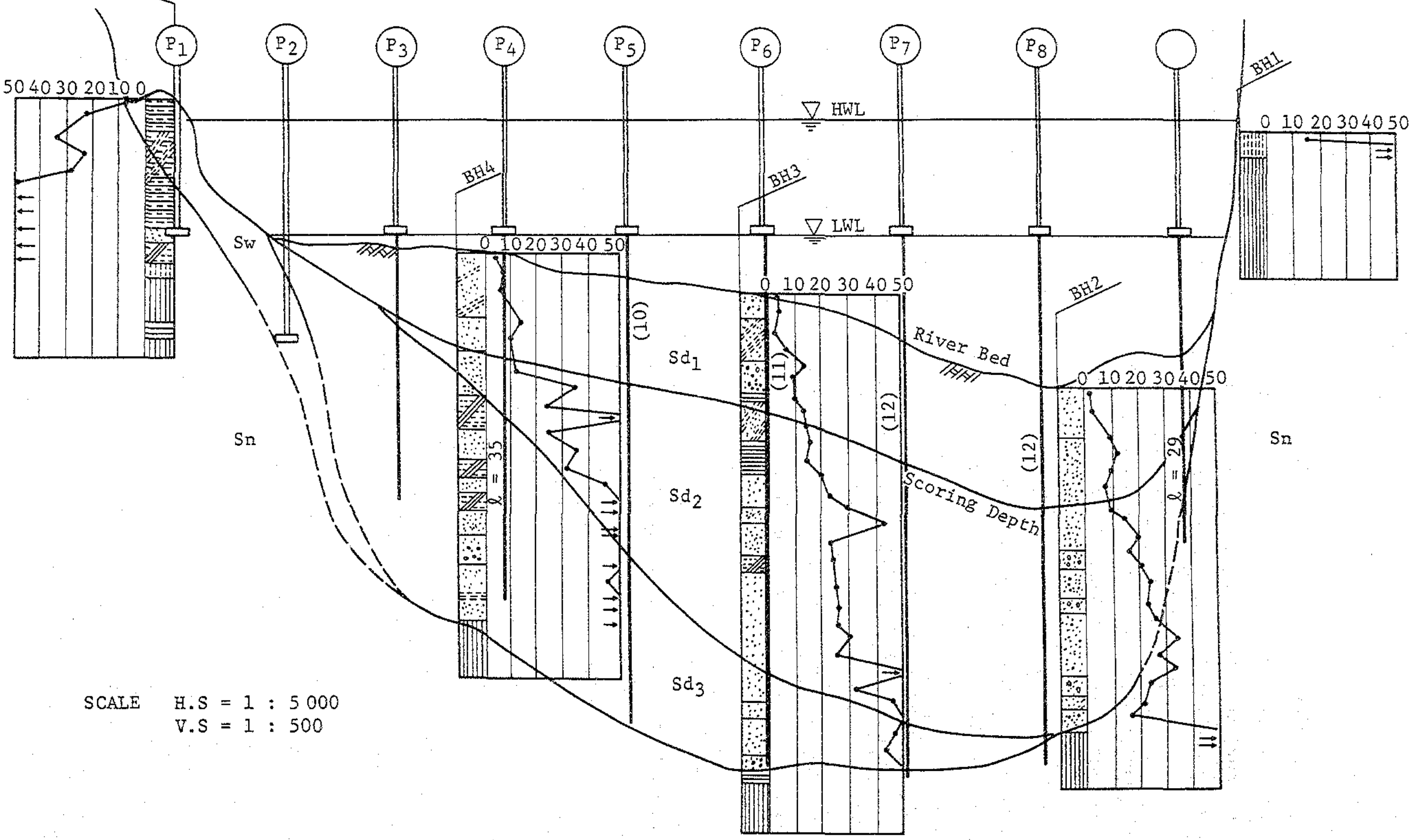
As the bearing stratum at piers P_3 to P_9 is deep as shown in Fig. 8.3.3.1, large diameter concrete pile bent foundations are adopted. For the shallow stratum where the rest of the piers (P_1 to P_2) and the abutments (A_1 & A_2) exist, conventional direct spread foundations are used.

In the pile bent foundation the piles are joined to their pile caps above the water level in order to avoid underwater construction work. As the construction of the pile has to be completed in the dry season the height of the pile caps are set at 20 meters above sea level considering the varying river water level in the 6 month dry season.

In pile bent foundations with long projection piles above the river bed, the number of these piles is governed by the horizontal force. In case of this bridge, the pile projections are quite long (cf. Appendix Table 8.3.3.1 and Appendix Table 8.3.3.2) considering scour depth. Earthquake action has the biggest effect to the horizontal forces.

As the biggest stress in a pile section occurs at the joint to the pile cap, the joint is reinforced preparing a taper. Diagrams of resistant stress resultants (M & N) are shown in Appendix Fig. 8.3.3.1.

Fig. 8.3.3.1 Soil Condition and Pile Length



The number of piles is determined in the way that the reaction of piles and the stress in pile sections due to the load are within allowable value (cf. Appendix Fig. 8.3.3.1).

The results of the analyses are as follows:

- Road Bridge:

  Appendix Fig. 8.3.3.2
  Appendix Table 8.3.3.2 - 8.3.3.12

- Rail-cum-Road Bridge:

  Appendix Fig. 8.3.3.3
  Appendix Table 8.3.3.13 - 8.3.3.24

8.3.4 Geometric Alignment

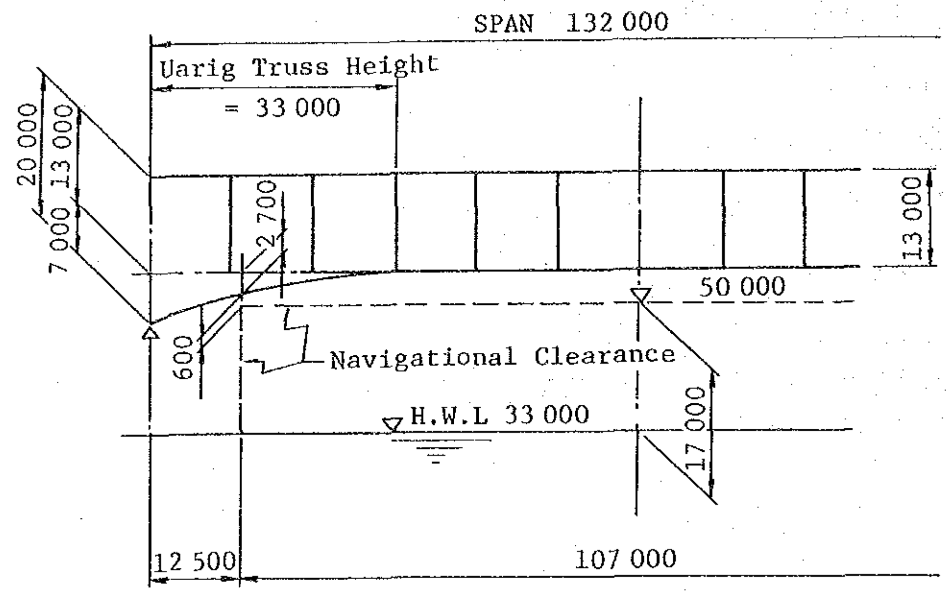
The horizontal and vertical alignment of this bridge is a straight line. The proposed elevation of bridges were determined so as to meet the requirement for navigational clearance as follows:

- Road Bridge : cf. Fig. 8.3.4.1

- Rail-cum-Road Bridge : cf. Fig. 8.3.4.2

Fig. 8.3.4.1 PROPOSED HEIGHT (TRUSS)

Profile



Cross Section

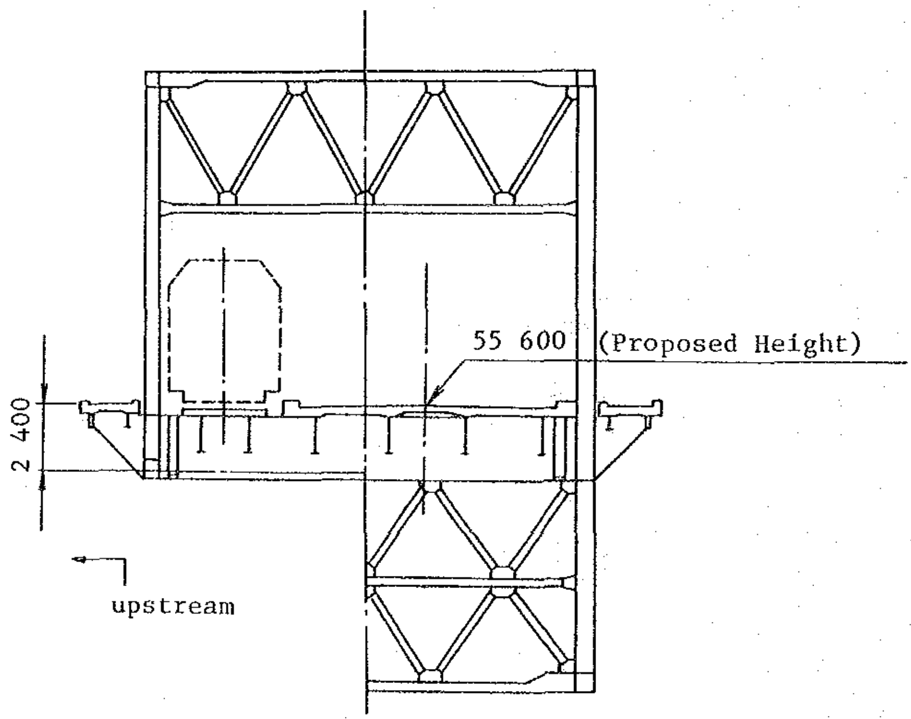
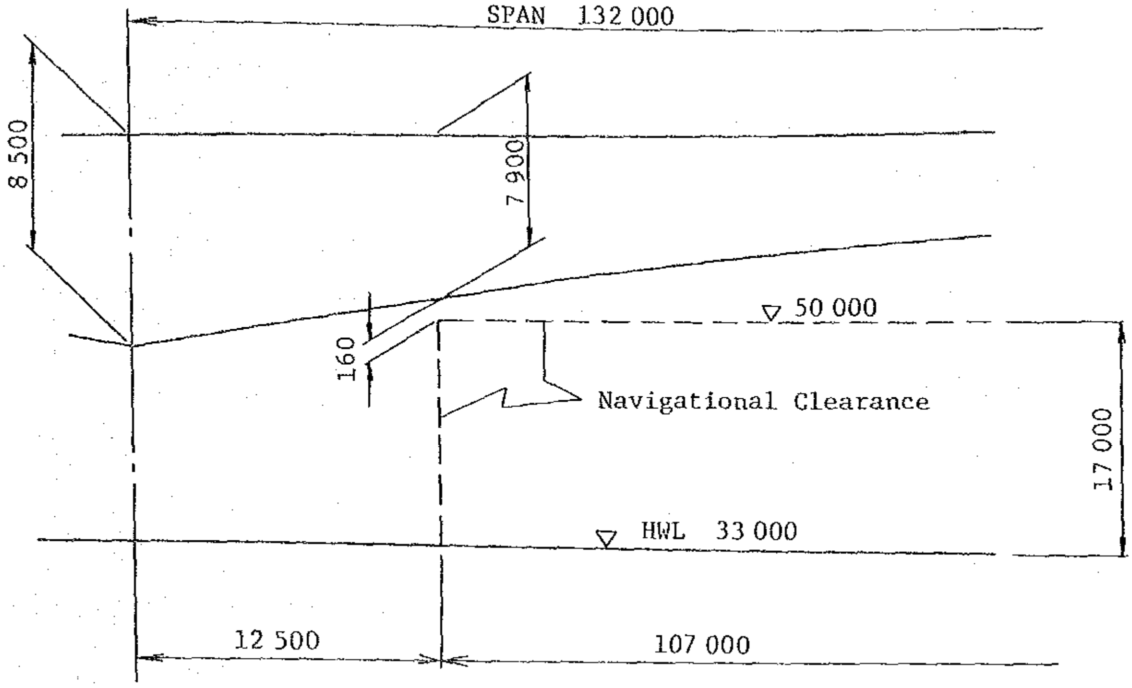
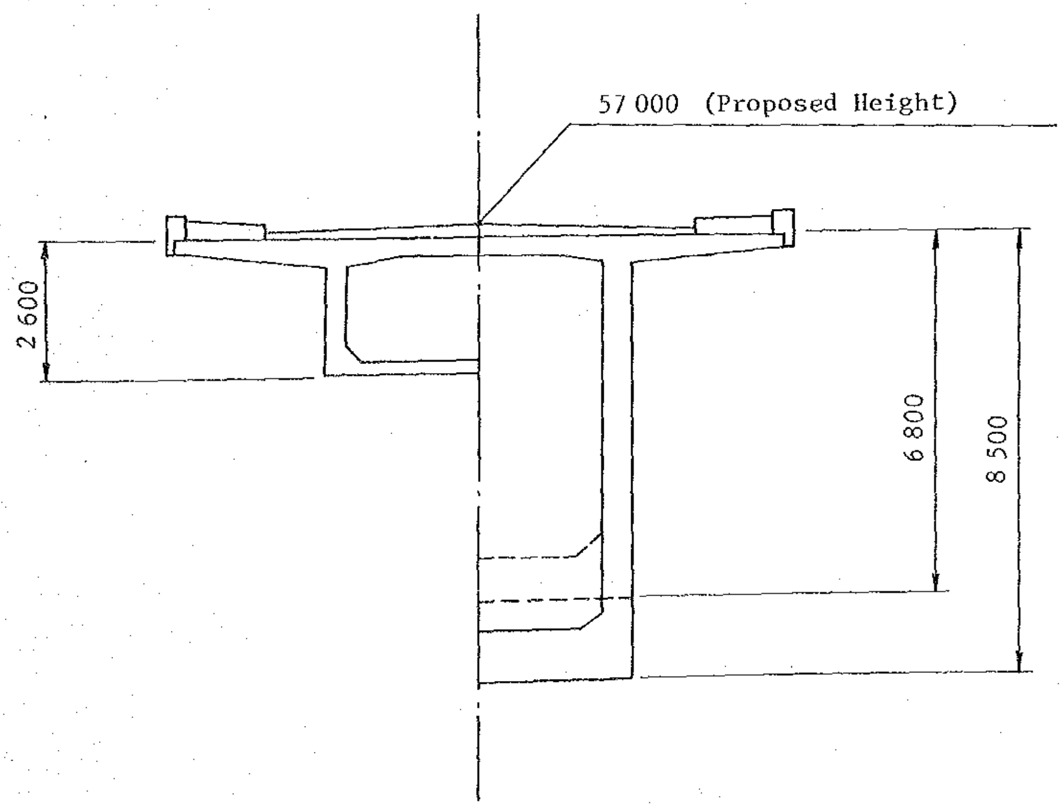


Fig. 8.3.4.2 PROPOSED HEIGHT (PC-BOX)

Profile



Cross Section



8.4 Construction Plan

8.4.1 General

This section describes the construction plan for the two bridges proposed (the road bridge and rail-cum-road bridge) concerning the facilities, construction method and schedule.

This information refers to the sections regarding on cost estimates and implementation described in this document.

(1) Construction Schedule

Due to the seasonal variation in water depth as shown in Fig. 9.2.1, construction of the substructure is limited to the 6 months of the dry season, i.e. from December to July. The construction of the substructure therefore needs to be concentrated throughout this limited period, necessitating day and night shift.

Special note should be taken of the potential long lead time required for preparation and mobilization of construction equipment to proposed site. Our investigation indicates that the preparation and mobilization period for special plant requires 9 months hence delays to construction schedule can be envisaged if contract is not awarded to commence at the end of rainy season.

In considerating above factors, two proposed bridges are planned for completion within four years (48 months) as shown in Fig. 8.4.1.1 & 8.4.1.2. It is stressed that the construction work would be delayed by 6 to 10 if the starting time does not coincide with the plan mentioned above.

(2) Construction Method

In considering the environmental and hydrological conditions prevalent at the proposed site we recommend that large floating barges be utilized for construction work.

For construction of concrete piles, the RCD (reverse circulation drill method) is adopted.

Fig. 8.4.1.1 CONSTRUCTION SCHEDULE (PC-BOX)

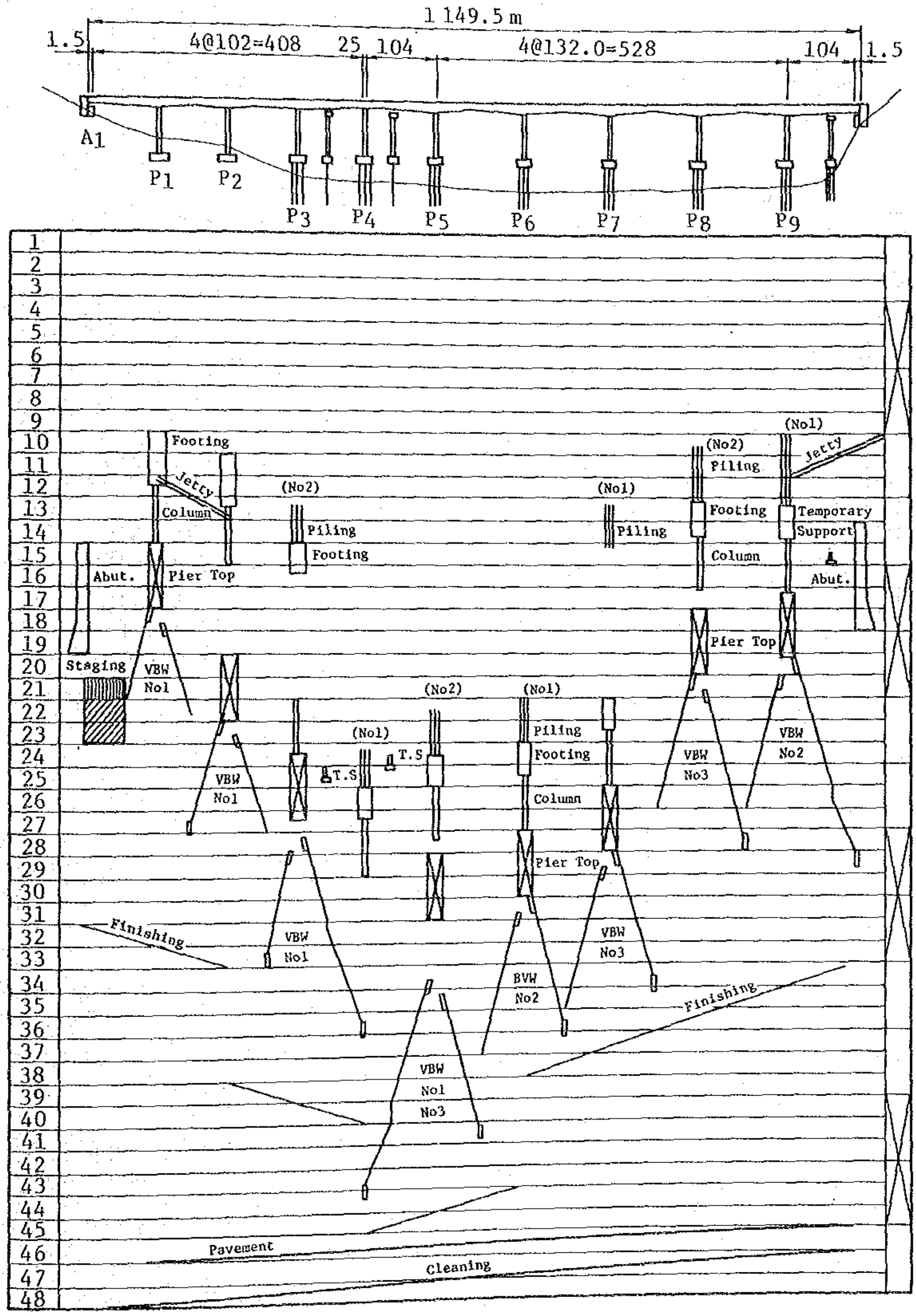
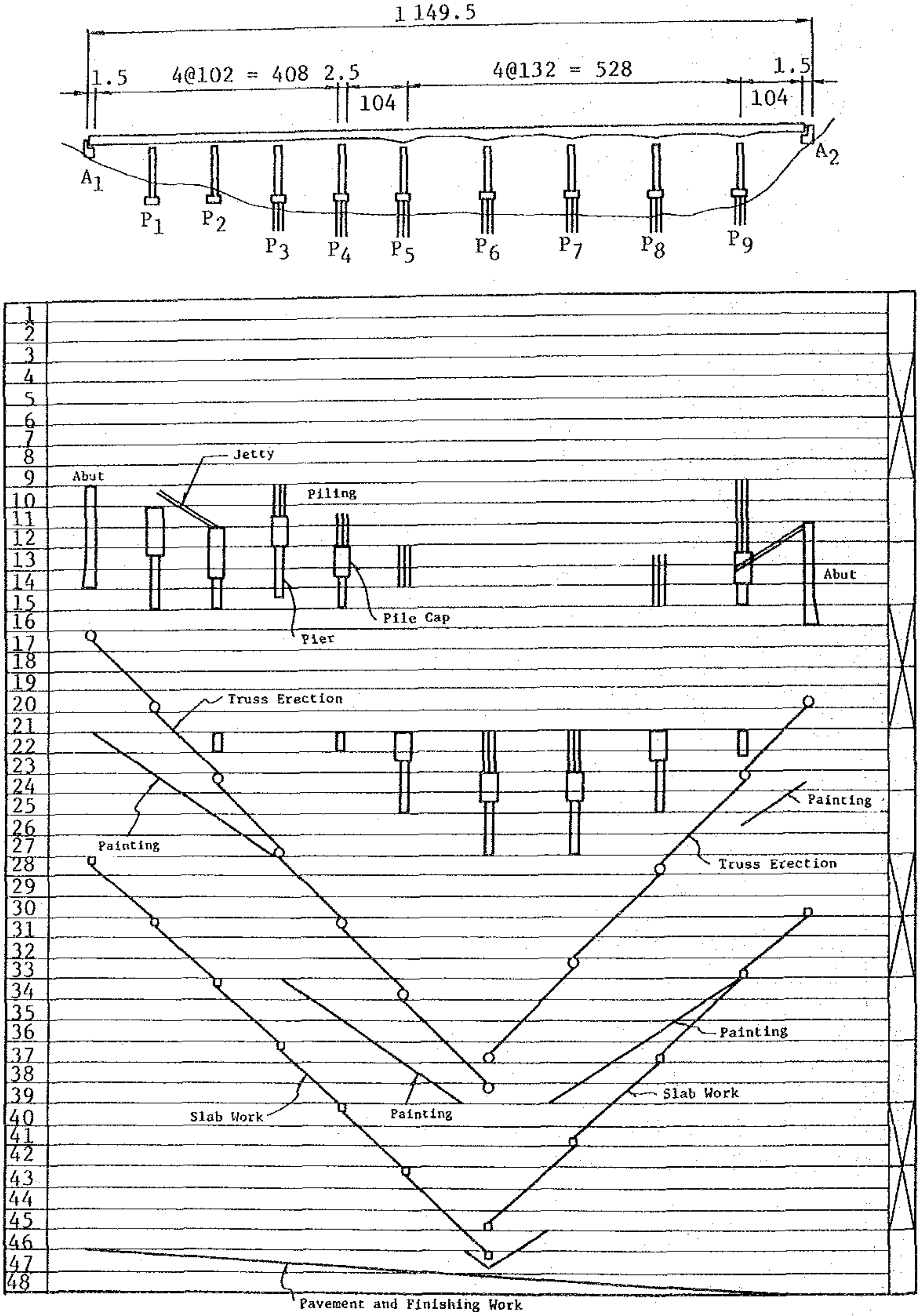


Fig. 8.4.1.2 CONSTRUCTION SCHEDULE (STEEL TRUSS)



For the superstructure constructions of both the road bridge and the rail-cum-road bridge the cantilever erection method is adopted.

(3) Inland Transportation

As the imports of main construction materials and heavy machinery is substantial, they will be transported by ship and/or by truck from Rangoon to the bridge site, utilizing local vessels and/or trucks.

(4) Temporary Facility

The temporary facilities are as described in section 8.4.3 below.

(5) Local Material

This study does not envisage importing of any materials available locally such as petrol, sand, gravel, timber, cement, etc. Note that delays in supply of such local materials would adversely impact construction schedule.

Most machines are assumed to be imported. Some machines in Burma are considered usable in the construction work. They are shown in Appendix Tables 8.4.4.1 - 8.4.4.4.

8.4.2 Temporary Facility Planning

(1) Land for Construction

Note it has been assumed in this study that all necessary land acquisition and permission for all construction work activities including base camp facilities would be fair issued to contractor (cf. Appendix Table 8.4.2.1.).

(2) Temporary Roads for Construction

In addition to above point (1) it is assumed contractor would have unrestricted use of the existing road for fertilizer plant from Prome Road to the plant.

(3) Jetty and Working Platform

Two jetties will be prepared at both banks on the downstream side and the seven working platforms at Piers (P_3 to P_9) as shown in Appendix Fig. 8.4.2.1 and 8.4.2.2 respectively, for loading and unloading construction goods, and as working platforms for construction areas in the river and for storage.

(4) Construction Base Camp

A base camp, for workforce including the construction office, working yard will be constructed on the east bank, the storage yard and warehouses on the west bank.

(5) Water Supply

For construction purposes, sedimentation concrete tank is constructed in the construction base camps on bothsides of the river.

(6) Electricity

Electric Power for construction work is envisaged to be supplied by installing temporary generators at site.

(7) Plant for Sand and Gravel

In the main base camp on the east bank, sand and gravel plants consisting of an aggregate screening machine (Cap 1.5 m x 3.0 m) and a sand washing plant (cap 30 tons/hours) are prepared.

(8) Concrete Batching Plant

A concrete batching plant (Cap. 60 m^3/hr) (cf Appendix Fig. 8.4.3.3 and 8.4.3.4) is envisaged to be set beside the plant for sand and gravel. All testing and storage facilities are provided at batching plant and/or main camp.

(9) Casing Rolling Plant

From an economics point of consideration casing rolling fabrication will be performed at site.

(10) Communication

It is assumed telephone lines are readily available at site.

8.4.3 Construction Plan of Substructure

The recommended procedure for the substructure is as shown in Fig. 8.4.3.1. Except for some differences in pier shapes and quantities of materials the span arrangements and substructure of two bridges (road bridge and rail-cum-road bridge) are the same. Their construction plans are explained as follows:

(1) Construction of A_1 & A_2 Abutments

The construction procedure is as shown in Fig. 8.4.3.2-(1). The key-elements of construction are as follows:

- Levelling by concrete
- Concrete building
- Placement of reinforcing bars for footing
- Placement of formwork for footing
- Placement of concrete for footing
- Abutment construction

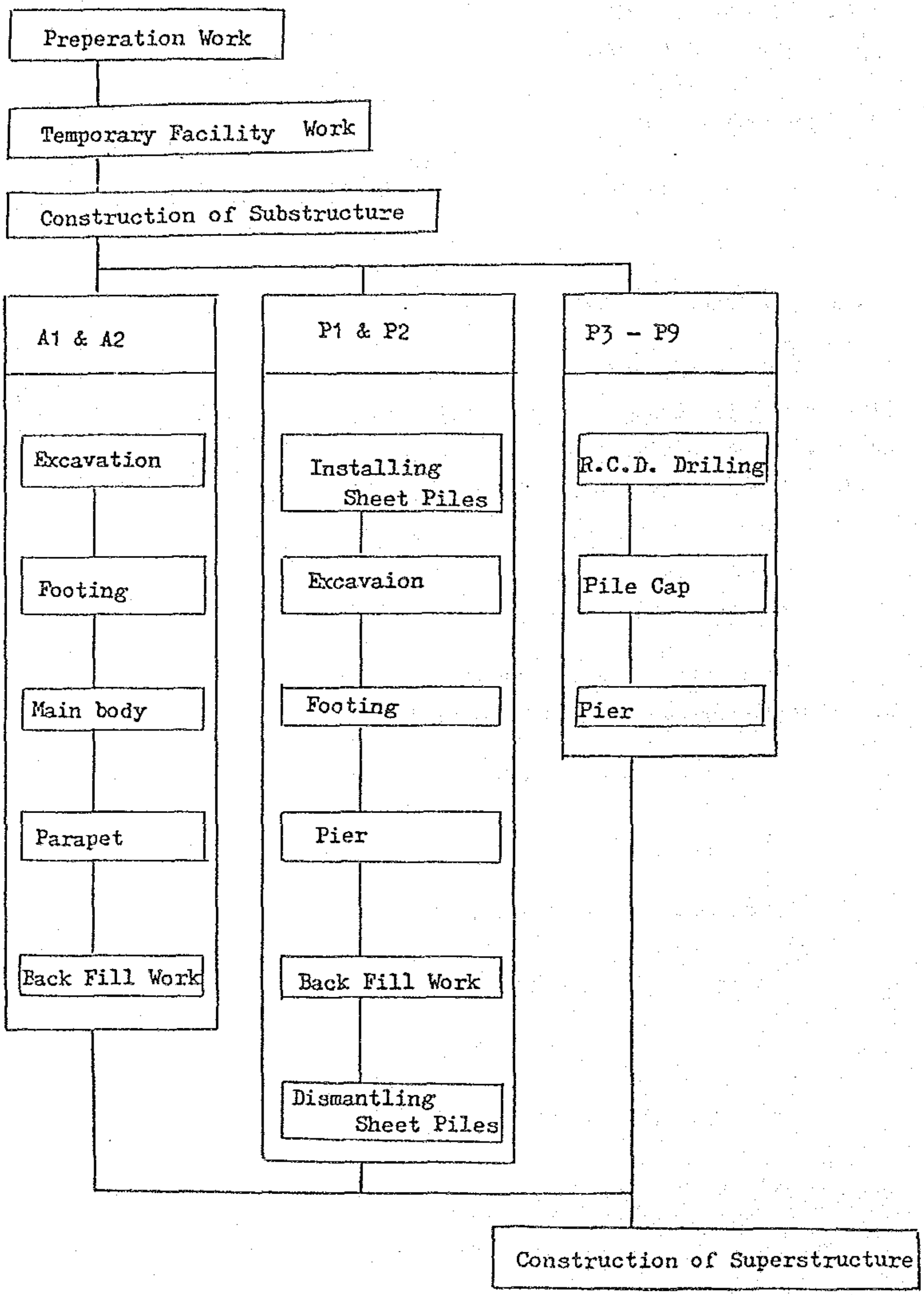
(2) Construction of P_1 and P_2 Piers

The construction procedure is shown in Fig. 8.4.3.2-(2).

The major activities are as follows:-

- Sheet pile work

Fig. 8.4.3.1 WORK FLOW OF SUBSTRUCTURE CONSTRUCTION



As the foundation height is under the ground water level, for its construction waterproof sheet piles (YSP III) with specified corner sheet piles (cf. Appendix Fig. 8.4.3.3) are provided.

- Excavation and finishing work

- Construction of footing

- Construction of pier

  It is envisaged that pier will be constructed in 5 meter pours from the ground level.

(3) Construction of P_3 to P_9 piers

The construction procedure is as shown in Fig. 8.4.6.2-(3)
The major activities are as follows:-

a) Piling work

The construction method will be the reverse circulation drill (RCD) method. In order to meet the construction schedule as detailed in this study it is necessary to use three sets of drilling machines to complete the work in two dry seasons.

Pile sketches and construction procedure are shown in Appendix Fig. 8.4.3.4 and 8.4.3.5, respectively.

One set of the machinery for the deep river is as follows:

o Barges for river work: RCD barge, bentonite mud pit barge, freight barge, and anchor barge.

o Engined ships: tug boat, accompanying ships, concrete carrying ships, and communication ships.

o Machines on the RCD barge:
Set of RCD machines, crawler crane, vibro-hammers, and generators.

o Machines on the mud pit barge:
mud screen with a cyclone and
bentonite mixer

Fig. 8.4.3.2 CONSTRUCTION FLOW OF EACH SUBSTRUCTURE

(1) Abutment

Footing
- Excavation
- Leveling by Concrete
- Setting R.bars
- Form Work
- Concreting

Main Body
- Setting Scaffold
- Setting R. Bars
- Form Work

Parapet
- Setting R. Bars
- Form Work
- Concreting
- Removing Scafold
- Filling back Earth

(2) P1 & P2

Footing
- Setting Sheet Piles
- Excavation
- Leveling by concrete
- Setting R.bas
- Form Work
- Concreting

Pier
- Setting Scaffold
- Setting R.bars
- Form Work
- Concreting
- Removing Form & Scafold
- Filling back Earth
- Removing Sheet Piles

(3) P3 - P9

Scafold Work
- Setting Brackets
- Fixing Studs
- Setting Beams
- Setting Plates
- Setting R.bars
- Setting Tem.Slab R.bars
- Concreting Temp Slab

R.C.D. Pile Work

Pile Cap
- Setting Lower R.bars
- Setting 1st Form
- 1st Concreting
- Setting Upper R.bars
- Setting 2d Form
- 2d Concreting

(4) R.C.D. Pile

- Procssing Casing Pipes
- Setting Pipe Holder
- Standing Casing Pipes
- Cram Shell Excavation
- R.C.D. Drilling
- Mesuring
- Standing 1st R.bars
- Inserting Tremie Pipe
- Slime Extraction
- 1st Concreting
- Cutting away Stand Pipe
- Pitching Concret Surface
- Standing 2d R.bars
- 2d concreting
- Leitance Extraction

R.C.D. Pile work procedure is as shown in Fig. 8.4.3.2-(4), and the major activities are as follows:-

- Processing casing pipes

  The casing pipes will be processed in the camp. Each pipe has a trumpet shaped taper at the top with a projected stand pipe as shown in Appendix Fig. 8.4.3.4.

- Casing Holder

  To set the casing pipes accurately, a casting holder will be prepared as shown in Appendix Fig. 8.4.3.6.

- Installation of casing

  The casing pipes will be vertically installed into the river bed through the casing holder by vibro-hammer.

- Clamshell bucket excavation

  If upper level of the pipe is filled with river bed soil then top eight metres will be excavated using a clamshell bucket.

- Drilling work with RCD

- Installation of outside reinforcing bars

- First-stage concrete

  It is required that first-stage concrete is placed up to the level of the bottom of the inner reinforcing bars which is installed later. Concrete is placed by tremie. As this concreting work is carried out in the hot dry season and takes a long time a retarder would be added into the mixed concrete.

- Treatment of joint concrete

  As the quality of the top portion of concrete is expected to be poor, the bentonite mud water is pumped out and laborers excavate by hand until they find sound concrete. The projected stand pipes will be cut away to be used again.

- Installation of inner reinforcing bars

- Second stage concrete

  As there is no water inside of the casing, the concrete will be placed in ordinary way using vibrators.

b) Construction of pile cap

- Scaffold work

  As shown in Appendix Fig. 8.4.3.7, scaffold for the footing shall be assembled in a grid-shape space.

- Erection of formwork

- Placement of concrete and rebars

c) - Construction of pier column

  Pier column will be constructed up to high water level prior to commencement of rainy season to allow work to continue during rainy season.

8.4.4 Construction Plan of PC-Box Girder

The superstructure will be constructed by the cantilever erection method except side spans and center segments. For the side spans between P4 - P5 and P5 - P6, there will be adopted two temporary bents to avoid large imbalance moments at P4 and P6 during the construction. The end portions of side spans at A1 and A2 will be constructed using scaffolds from the ground and the center segments using suspention scaffolds.

The construction would be carried out in accordance with the chart in Fig. 8.4.4.1.

The major work activities are as follows:-

Fig. 8.4.4.1 CONSTRUCTION FLOW OF PC BOX GIRDER

General Work Flow

Pier Capital

Segments on Cantilever

Center Hinge Segment

Bridge Surface

Segment on Staging

Setting Scaffold

Lower Slab
- Form Work
- Setting Rebar
- Setting PC-bars
- Concreting

Web & Upper Slab
- Form Work
- Setting Rebar
- Concreting

Stressing PC-tendons

Removing Form & Scaffold

Center Hinge Segment

Setting Scaffold
Form Work
Setting Scaffold
Setting Rebar
Setting C. Hinge
Setting PC-Tendons
Concreting
Removing Scaffold

Pier Capital

Lower Slab
- Form Work
- Setting Rebars
- Setting PC-Bars
- Concreting

Webs
- Form & Scaffold
- Setting Rebars
- Setting PC-Bars
- Concreting

Upper Slab
- Form & Scaffold
- Setting R. Bars
- Setting PC-Bars
- Concreting

Stressing PC-Tendons

Removing Scaffold

Segments on Cantilever

Setting Wagen

Form Work
Setting Rebar
Setting PC-bars
Concreting
Stressing PC-Tendons
Shifting Wagen

Finish Segment Work ?

No

Yes

Removing Wagen

(1) Pier Top

- Scaffold

  Refer to Appendix Fig. 8.4.4.1 & 8.4.4.2.

- Formwork

- Arrangement of reinforcing bars and PC tendons

  For detail of PC tendons (SBPR 95/120 ∅32) covered by sheaths refer to Appendix Fig. 8.4.4.3.

- Concreting

- Prestressing

  PC tendons will be stressed to the specified tension by D&W jacks. The prestress give the pier top enough strength to set up Frei Vorbau Wagens.

(2) Cantilever Construction

A Frei Vorbau Wagen (cf. Appendix Fig. 8.4.4.1 & 8.4.4.2) will be installed on the pier top after construction of the first segment of the girder. Another Wagen will also be installed on the opposite side of the first segment. The second segment is constructed. The girder will then be constructed segment by segment using the two Wagens alternately. The cycle time of the work, which consists of formwork, an arrangement of reinforcing bars and PC tendons, concreting work, dismantling of forms, prestressing and shifting the Wagen will be 12 days, as normal cement is used in this girder (cf. Table 8.4.4.1).

In construction of the side span at P5 pier, temporary bent will be used along with the Wagen. The bent will be installed using the barge.

(3) Staging Construction

For details refer to Appendix Fig. 8.4.4.4.

(4) Center hinge segment construction

Each center segment will be constructed on the suspended scaffolding (refer to Appendix Fig. 8.4.4.5). The scaffold will be suspended from the end of the constructed cantilever girder by deformed PC tendons.

8.4.5 Construction Plan of Steel Truss

In the case of a steel truss bridge construction all the members of the superstructure will be fabricated by experienced fabricator and assembled on the site.

The erection (cf. Appendix Fig. 8.4.5.1) will proceed from both ends of the bridge to the center. The key-elements are as follows:

(1) Fabrication

In order to reduce the reaction from the superstructure to the substructure, the weight of steel raw material will be minimized by using high tensile strength steel of SM58 for the main structure of the bridge. This fabrication work will be performed by experienced fabricator.

(2) Transportation of Components

The only wharf in Rangoon Port equipped with a 40 ton crane is the Brooking Street Wharf No. 1. On all other wharfs the maximum capacity of cranes is 3 tons. As the heaviest bridge components would be from 7 to 8 tons in weight an additional movable crane of 45 ton capacity would be provided if any of these wharfs are appointed.

It is assumed for this study that local barges and tug boats will be available in Burma to transport components to site in order to meet the erection schedule.

(3) Erection

- Selection of erection method

In considering the environmental and hydrographical conditions at site, navigation requirements, construction period/season, etc., it will be required to construct the bridge without using any supports erected in the river, and the following erection methods, which don't need scaffoles, are selected for investigation in this study;

- Large block erection method
- Incrementally launching
- Cantilever method

As a result of our investigation the cantilever erection method was selected as being the most suitable from execution point of view as large lifting cranes necessary for block erection method are not available in the area and the large quantity of equipment/machinery and labour required for incremental launching method is not suitable from economical and schedule point of view.

- Flow chart of erection work

Erection work shall be performed following the flow chart shown in Fig. 8.4.5.1. The work shall be started from both sides of the river at the same time.

- Erection of spans

Erection of spans will be performed in the conventional erection method. For details of bents and steel tower refer to Appendix Fig. 8.4.5.2 and Appendix Fig. 8.4.5.3 respectively.

Each panel of the truss will be completed following the flow chart in Fig. 8.4.5.2.

(4) Placement of floor slab

Major work activities are as follows:-

- Erection of form work

- Placement of rebar

- Placing of concrete

Fig. 8.4.5.1 WORK FLOW CHART

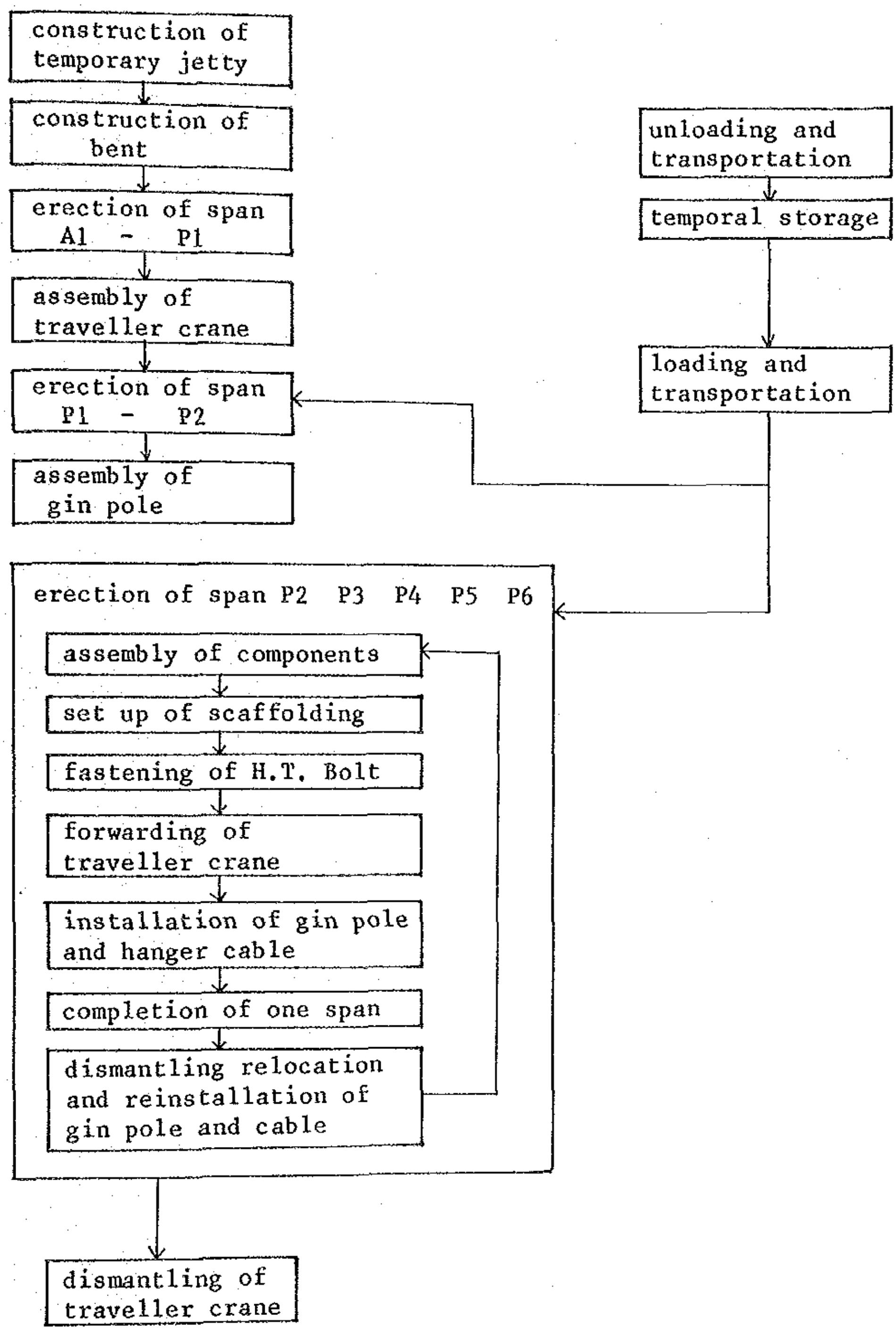


Fig. 8.4.5.2 ERECTION WORK IN A PANEL