REPORT ON INVESTIGATIONS FOR CONSTRUCTION OF THE GORAI RIVER BRIDGE EAST PAKISTAN

DECEMBER 1967

OVERSEAS TECHNICAL COOPERATION AGENCY

GOVERNMENT OF JAPAN

PREFACE

The Government of Japan, at the request of the Pakistani Government entrusted the Overseas Technical Cooperation Agency with conduction of a feasibility suvey for the Gorai River Bridge Construction Project in East Pakistan.

Therefore, the Overseas Technical Cooperation Agency dispatched survey missions headed by Mr. Nobutaka Katahira, director of Japan Highway Public Corporation , to conduct the survey in three stages from1966 to 1967, being fully aware of the significance of the mission for the economic growth of the country caused by the construction of the Gorai River bridge.

The investigations conducted at this time were to select the bridge and to perform the preliminary and comparative designing for the proposed Gorai River bridge. The results of the field investigations have been reviewed and compiled into the Report that is hereby submitted to your Government.

Nothing would be more gratifying to our Agency than if this report could be of any use in the promotion of the project in your country as well as in the furthurance of the amity, friendship and economic relations between Pakistan and Japan.

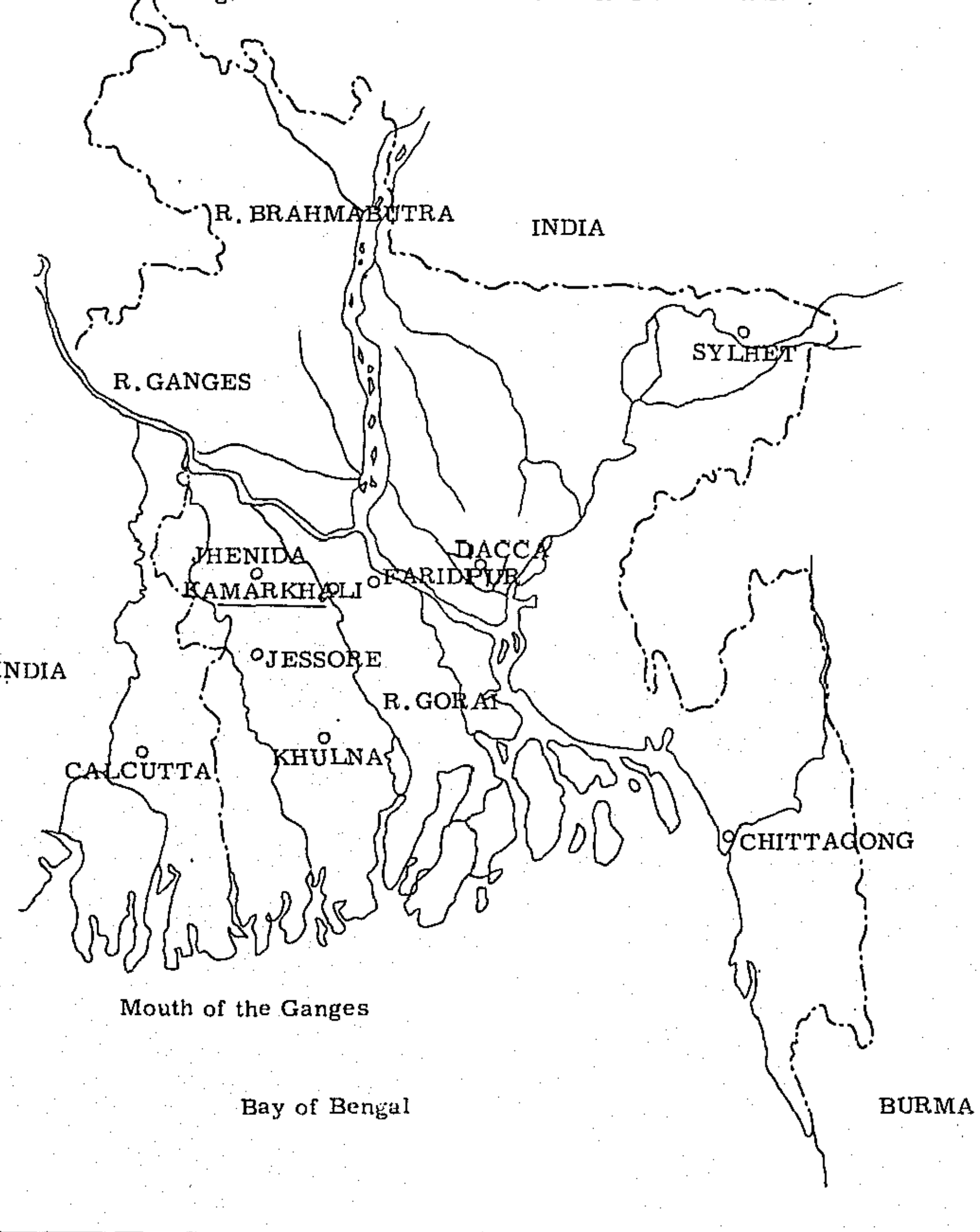
We deeply appreciate the favours granted by the Government of Pakistan and it's agencies who have given their hearty assistance and cooperation for execution of investigations during the stay of the missions in Pakistan.

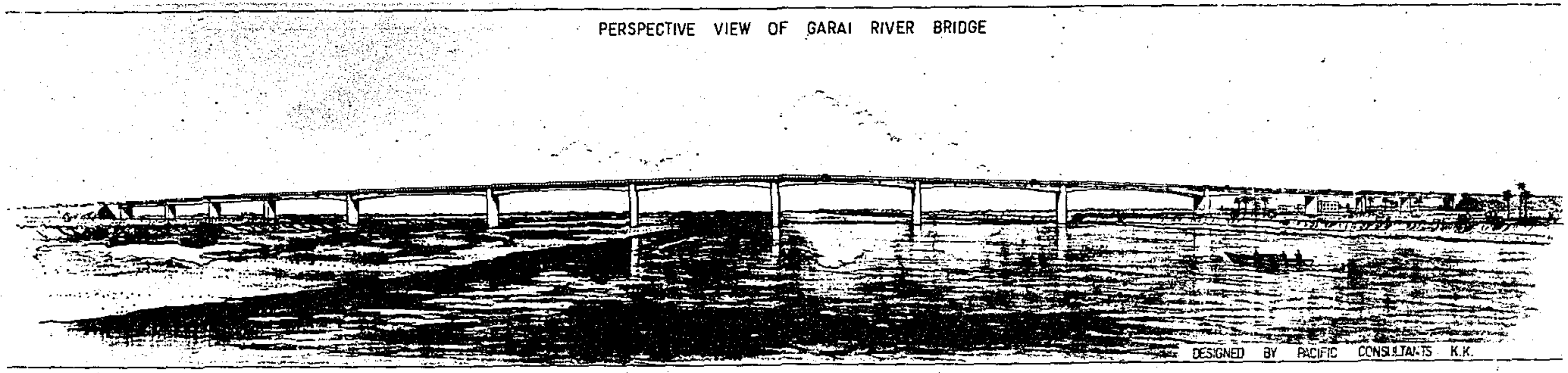
December 1967

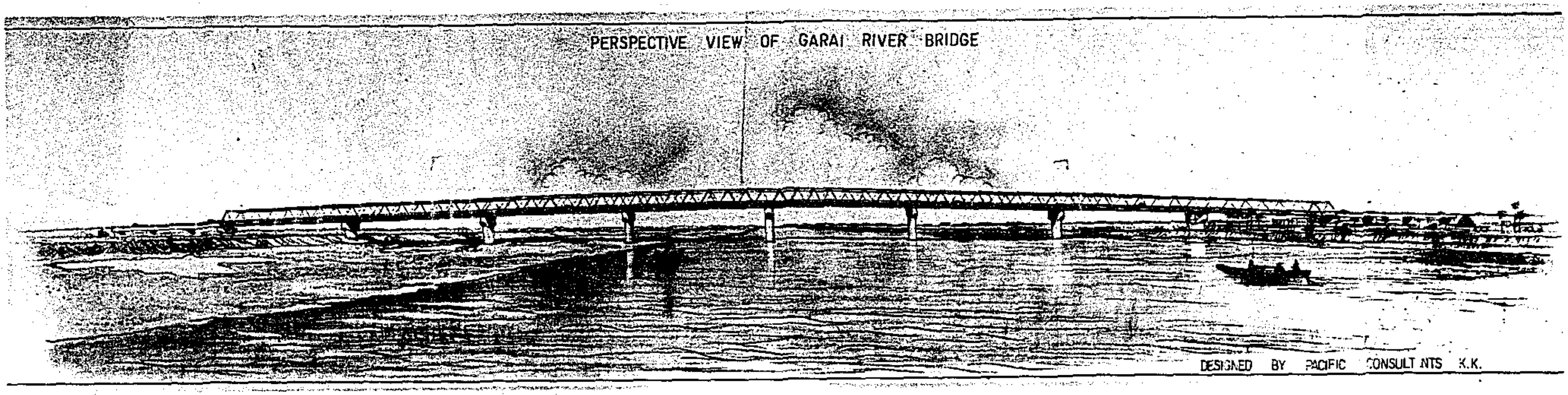
S. Shibusawa

Shinichi Shibusawa
Director General
Overseas Technical Cooperation Agency

Fig. 1 LOCATION MAP OF EAST PAKISTAN




PERSPECTIVE VIEW OF GARAI RIVER BRIDGE
DESIGNED BY PACIFIC CONSULTANTS K.K.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CHAPTER I INTRODUCTION

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1. General

The Overseas Technical Cooperation Agency having reached the decision in March 1966 that it would undertake investigations for the feasibility of the Gorai River Bridge Construction in East Pakistan, organized a survey mission headed by Mr. Nobutaka Katahira, Director of Japan Highway Public Corporation (NIHON DORO KODAN).

The survery mission subsequently carried out comprehensive investigations for the proposed project.

Investigations were carried out in three phases each a different survey. The preliminary survey which constituted the first phase survey lasted from late March to early May, 1966. The Survey taken during the rainy season which was the second phase survey was performed during the period from late August to early September of the same year. The third pahse survey including soil exploration, economic and material study was conducted from the latter part of December 1966 to the middle of March, 1967.

The results of the investigations have been compiled and are presented herein in a comprehensive form, together with alternative designs for the proposed bridge as prepared on the basis of technical studies of the survey results.

The first half of this report deals with different field surveys and the collection of various data and materials required for mapping out the project. The latter half, in which is described the outcome of technical analysis of data and materials as well as of field surveys, contains preliminary designs of the bridge based on such analysis, estimation of cost, economic feasibility of the project, and opinions from the technical standpoint in respect to the proposed construction work.

The first phase survey, which was the basic field survey, was carried out mainly for the selection of the bridge sites and topographical survey.

Formation of the first phase survey mission and duration of its survey activities are given below.

Formation:

Leader	Mr. Nobutaka Katahira	(Japan Highway Public Corporation)
Member	Mr. Takahiko Hayanari	(")

Member	Mr. Hideo Chiba	(Pacific Consultants, K.K.)
"	Mr. Nobutoshi Aijima	(")
"	Mr. Shigenobu Matsuda	(")
"	Mr. Shoichi Kamayachi	(")
"	Mr. Kenji Tanaka	(")
"	Mr. Masao Kuwabara	(Overseas Technical Cooperation Agency)

Duration of survey activities:

March 29, 1966 to May 3, 1966.

The second phase survey was carried out chiefly for the flood survey during rainy season in East Pakistan and for clarifying general and hydraulic conditions at the proposed bridge sites.

Formation of the Second Phase Survey Mission and the duration of its survey activities are as given below.

Formation:

Leader	Mr. Nobutaka Katahira	(Japan Highway Public Corporation)
Member	Mr. Shigeyuki Watanabe	(River Dept., Ministry of Construction)
"	Mr. Shigenobu Matsuda	(Pacific Consultants, K.K.)

Duration of survey activities:

August 29, 1966 to September 11, 1966

The third phase survey was performed mainly for soil exploration and geological survey at the bridge sites by means of boring, economic study, and investigations of local construction materials.

Formation of this Survey Mission and the duration of its activities are as given below.

Formation:

Leader	Mr. Nobutaka Katahira	(Japan Highway Public Corporation)
Member	Mr. Jun Mise	(")
"	Mr. Hideo Chiba	(Pacific Consultants, K.K.)
"	Mr. Shigenobu Matsuda	(")
"	Mr. Masao Kuwabara	(Overseas Technical Cooperation Agency)
"	Mr. Hiroyoshi Oyama	(")

Member	Mr. Kiyomi Suzuki	(Kajitani Chosa Koji K.K.)
"	Mr. Ichinoshin Nishimura	(")
"	Mr. Norio Kashiwagi	(")
"	Mr. Nobutoshi Kamiie	(")
"	Mr. Ikuo Terui	(")

Duration of survey activities:

December 10, 1966 to March 12, 1967

During the entire period of investigation, the Mission was given most valuable assistance and advice by Mr. Nishikawa, Consul-General, Mr. Oshio, Consul, Mr. Matsumoto, Vice-Consul, of the Japanese Consulate-General in Dacca and by Mr. M. Karim, Secretary of the Consul-General and further, by unlimited cooperation by officials in charge of competent government authorities in East Pakistan.

On the occasion of presenting this Report, the Mission wishes to express its most sincere gratitude for the invaluable assistance rendered by the individuals listed below.

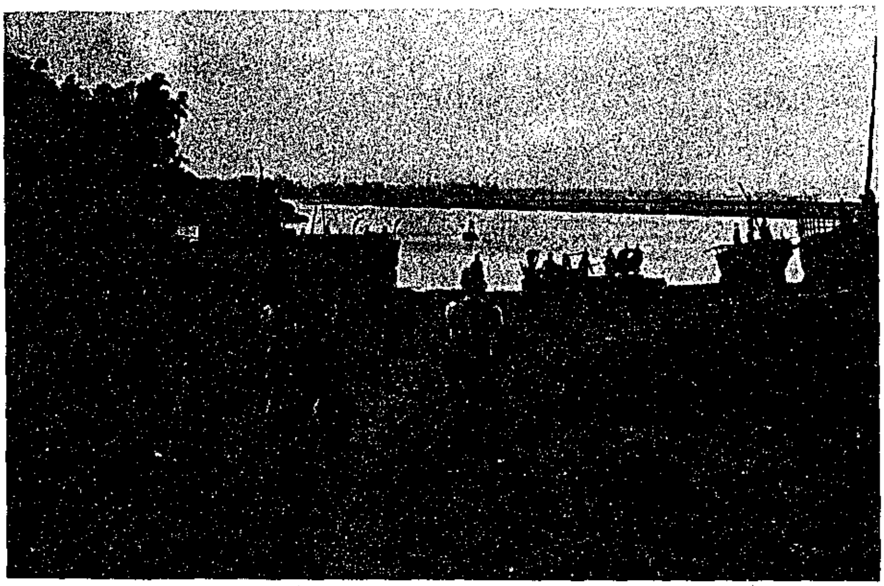
List of Name

Mr. H.T. Ali	Additional Chief Secretary Government of East Pakistan
Mr. H. R. Malik	Secretary, Railways, Waterways & Road Transport Department
Mr. Ramman Khan	Joint Secretary, Railways, Waterways & Road Transport Department
Mr. Shafiul Alam	Deputy Secretary, Ditto
Mr. H.A. Khan	Chief Engineer, Ditto
Mr. M.A. Zaman	Deputy Chief Secretary Ditto
Mr. A.R. Choudry	Executive Engineer, Ditto
Mr. W. Choudry	Executive Engineer, Ditto
Mr. M.A. Samad	Executive Engineer of Road Design I. Bengal Development Corporation
Mr. S. Hakimuddin	Executive Engineer of R & H, Faridpur

2. Characteristies of Surveys

The preliminary survey was carried out for the Gorai River Bridge Construction Project by the First Phase Survey Mission which was organized in March 1966. It involved investigation for the selecting of the bridge sites, comparative studies between the selected sites, topographical survey, river sounding and velocity observation at the selected sites, and reconnaissance of the route of the approach road.

These surveys and observations were conducted with the view of checking technical and economical feasibility of the Project. As a consequence, the conclusion was reached



The menhers of East Pakistan
with Japan survey team
(The first phase survey)

The view of
Kamarkhalighat

The Pakistan road
(A part of Asian Highway)

that the proposed project would inevitably require the surveys of river conditions during the rainy season peculiar to East Pakistan, a series of soil surveys, and investigation of the local construction materials.

The surveys covered a considerably long period of time as mentioned in the preceding section, and were carried out in three stages for various surveys including the above three items. The leader of the Mission did not fail to be at the survey site to control and grasp the entire view-point to arrive at the most comprehensive conclusion . Further Survey Missions of respective phases were organized by those experts capable of fulfilling the purpose of the survey.

Utmost attention was paid to meeting the requirements of the Government of Pakistan in the designing work which was executed with the prerequisite that the superstructure be confined to either a prestressed concrete bridge or a steel bridge, and that concrete prepared with crushed hard bricks as the coarse aggregate be used in constructing the substructure to make use of local materials as much as possible.

3. Outline of Design

The selection of the bridge site was conducted with due consideration given to the existing and future arterial road networks. Efforts were made to select the section of comparatively small errosion, small variation of river, and narrow width. Geological surveys such as boring works reveald that the selected site has sufficient strength for the foundation of a substructure with a long span.

From the results of hydraulic survey which was conducted twice, i.e. in dry and rainy seasons, the positions and spanning of bridge piers for the prestressed concrete bridge were determined by paying special attention to errosion of the river bank which is ascribable to the scheme of the pier arrangement. For the main water channel, 6 spans of 70 m, were arranged, and for either side of the river, 1 span of 50 m and 3 spans of 30 m were arranged. Although there are various conditions affecting the choice of bridge type, the most important prerequisites are the economy, construction period required, conditions of execution, maintenance, beautility, utility value, etc.

Regarding the economy, close examination and judgement is considered essential in respect to the ratio of local currency to foreign currencies, self-affordability and usability of local materials, quality and cost of labor available, etc.

It is recommended to utilize the local materials as much as possible with consideration given to the above cited points.

The design conditions adopted here are that there are small earthquakes, i.e., small or no effect due to horizontal force and that the temperature change is held within 40°C, *etc.*

Coarse gravel for concrete is not only hard to obtain locally but is also very expensive. Crushed stone should be used for the structure requiring high strength concrete, and crushed hard bricks may possibly be used for relatively low strength concrete required, as well as for the foundation, etc. By making allowances for the above conditions, two bridge types were selected and compared: one having a steel superstructure is of the continuous through truss bridge and the other is made of prestressed concrete (PC bridge).

In either case, for the principal part of the substructure, the concrete well foundation is adopted.

Although the steel truss superstructure was planned at the outset, the Mission has decided to employ the PC bridge of T- shapet statically determinate structure cantilevered by a 30 m PC precast girder. This was because of such factors as the foundation strength of as large as 70 t/m^2 found by the geological survey, (in the dense medium sand 30 m below the river bed), less frequent earthquakes, foreign currency situation and materials available in East Pakistan. This type of bridge is excellent in beautility and advantageous in maintenance.

On the other hand, the steel truss bridge was intended to be so designed that its structure is of through-bridge Warren type continuous truss in view of navigable clearance and that its principal parts use high tension steel (SM 50) for economy's sake. In East Pakistan, however, this kind of high tension steel is hard to obtain and must depend on import.

Results of squaring the construction costs on the above two types shows that either one of the two cannot be placed above the other in respect to the total cost (excl. compatibility of unit cost and rate of interests), and that the ratio of foreign currency to the local currency is 65: 35 for a steel truss bridge and 40:60 for a PC bridge, respectively. This indicates that the latter requires less foreign currency than the former.

Moreover, the adoption of PC bridge involves in itself that PC girder is useful from the viewpoint of bridge rectification policy , and it is desirable that the economical promotion of bridge construction through prehabilitation of P.C. girder could be obtained.

CHAPTER II FIELD SURVEY

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1. Preliminary Survey

The preliminary survey was carried out over in four stages as described below.

During the first stage survey, a number of consultations were held with competent authorities of East Pakistan to obtain the actual state of major roads, bridges and rivers existing in East Pakistan. Investigations were carried out with efforts directed towards collection of basic data and materials required for the propose of the construction of the bridge.

In the second stage, a number of bridge sites were first plotted on the map with consideration given to river conditions, road condition for connection with the proposed bridge, and the possibility of attaining the alignment which would form a part of the Asian Highway in the furture. Two such plotted sites were selected as prospective bridge sites after making field surveys by means of speed boats.

The third stage survey included field investigations and a topographical survey at the two selected sites.

Finally for the fourth stage survey, studies were made in respect to the survey items to be carried out for further execution of the project.

(A) Factors Affecting Selection of the Bridge Site

The following were taken into account as vital factors in selecting the two bridge sites in view of conditions peculiar to East Pakistan.

(1) The bridge site should be conveniently located for connection with the roads now in use as well as with the network of roads that may be constructed with future economic development.

(2) The bridge should be constructed, as far as possible, at a site which is free from erosion of banks and variation of river bed. This was considered necessary because of characteristics common to rivers in East Pakistan.

(3) The river should have as small a section and depth as can be attained at the bridge site. These two contradictory factors are to be carefully compared and reviewed before reaching final decision since smaller river sections

make the bridge length shorter, while a shallower depth is a factor demanded for easy construction.

(4) The topography of the site should be such that the minimum possible cost may be incurred by the construction of roads to be connected with the proposed bridge.

(5) The soil should have the nature and the suitable strength for the bridge designed.

(B) Selection of the Bridge Site

The proposed Gorai River Bridge was originally planned to be constructed at Kamarkhali located at the terminal point of the road which runs from Faridpur and is considered to constitute part of the future Asian Highway.

As had been expected, however, field surveys revealed that this originally planned point is located midway between severe bends of both upstream and downstream sides. These bends have been, and clearly tend to be, aggravated by floods that develop the erosion of outer banks and formation of sand-bars from inner banks. Kamarkhali is therefore considered not suitable for the proposed site of the Gorai River Bridge.

The arterial road which connects Jessore and Dacca, capital city of East Pakistan, runs through Faridpur after crossing the Gorai River by ferry boat, and runs further to cross the main stream of the Ganges. If, therefore, Kamarkhali is selected as the bridge site, no problem would arise in respect to the connection with the arterial road, in present. However, further examination disclosed that this route makes a detour passing through Jessore - Jhenida, and is not the shortest connection between Jessore-Faridpur - Dacca. This route was therefore considered not recommendable as the arterial road to be incorporated in the proposed project.

For the above-mentioned two reasons, the idea of constructing the bridge at Kamarkhali was dropped. This was followed by consultations with officials concerned of the Government of East Pakistan. As a result, reconnaissance was conducted on foot and by speed boats to the downstream of Kamarkhali. This reconnaissance resulted in the selection of three sites, at points 8 km, 10 km and 13 km downstream from Kamarkhali, which were found to be commendable from the viewpoint of water flow, condition of the banks, and connection with the approach road. By making further review of these sites, the first point, which is closest to Kamarkhali with a distance of 8 km, was precluded. This was because it is subjected to a heavier influence of floods, such as bank erosion and the changing in the figure of the river and river bed than the other two sites. The remaining two sites were considered to have the needed condition condition required for the proposed bridge site.

Field surveys carried out at these two sites included the determination of the center line of the bridge velocity and depth measurements of the river, and a topographical survey.

It may be mentioned that consulations held between the Survey Mission and the officials of the Highway Department of East Pakistan led to the latter's confirmation that the proposed sites amply satisfy various conditions required.

(C) Works at the Bridge Site

Following are the surveys carried out at the proposed bridge site.

(1) Selection of bridge site and determination of its center line.
(2) Triangulation of the present river width.
(3) Topographical survey of bridge site and its vicinity by plane-table and stadia survey.
(4) Installation of bench marks (with BN No. 101 of WAPDA at Kamarkhali adopted at datum level).
(5) Measurement of water depth directly below the center line of the bridge. (Sounding machine, Model P-612, manufactured by Japan Supersonic Inst. was used in measuring the water depth).
(6) Velocity measurement
Velocity measurement was carried out by means of bamboo buoys measuring 8 cm (dia.) x 80—100 cm (length). The bamboo buoys were filled with appropriate quantity of sand and arranged to float within a 200 m distance up- and down-stream of the planned center line of the bridge.

(D) Approach Road

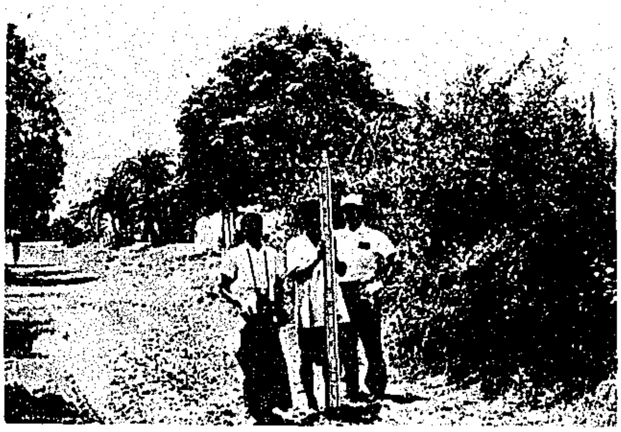
Since the field surveys were carried out mainly for the purpose of selecting the bridge site, studies pertinent to the approach road to the bridge were inevitably limited to a lesser degree, though they included location on the map and reconnaissance conducted based on such map locations. The alignment of the approach road, particularly when considered as a highway route, is one of the most important factors affecting the proposed site, and it necessitates detailed surveys prior to the actual execution of construction work.

The results of surveys made in relation to the approach can be summarized as follows.

Topographical survey
(The first phase survey)

Topographical survey
(The first phase survey)

Topographical survey
(The first phase survey)



Installation of bench
mark
(The first phase survey)

It is considered justifiable that the new route of the arterial road to be constructed in the future should directly connect Jessore and Faridpur. As a result of reconnaissance, therefore, the Mission selected the route shown in Fig. 2. This route is arranged to start from a point closer to Khulna, about 7 miles from Jessore City, and not to be directly connected with Jessore City, so that it may serve as part of the planned Asian Highway for an almost straignt connection with Calcutta. This route would also be advisable for easier access to Khulna.

If the Gorai River Bridge is completed before the construction of the above-mentioned arterial road, the following measures may be advisable to cope with the flow of traffic expected to arise when the existing road is connected with the completed bridge.

The road which extends between Binopedur and Magura which is located midway on the route of the arterial road running in the vicinity of the bridge site and connecting Jessore - Jhenida - Faridpur is to be improved and widened for connecting the Gorai River Bridge. The road thus widened and remodeled can not only be connected with the permanent arterial road to be constructed in the future, but also would be quite useful for the development of Jhenida District. Such a road would contribute in the ultimate to the establishment of a complete network of roads covering all these districts.

2. Hydraulic Condition of Bridge Site

(A) Outline of the Gorai River

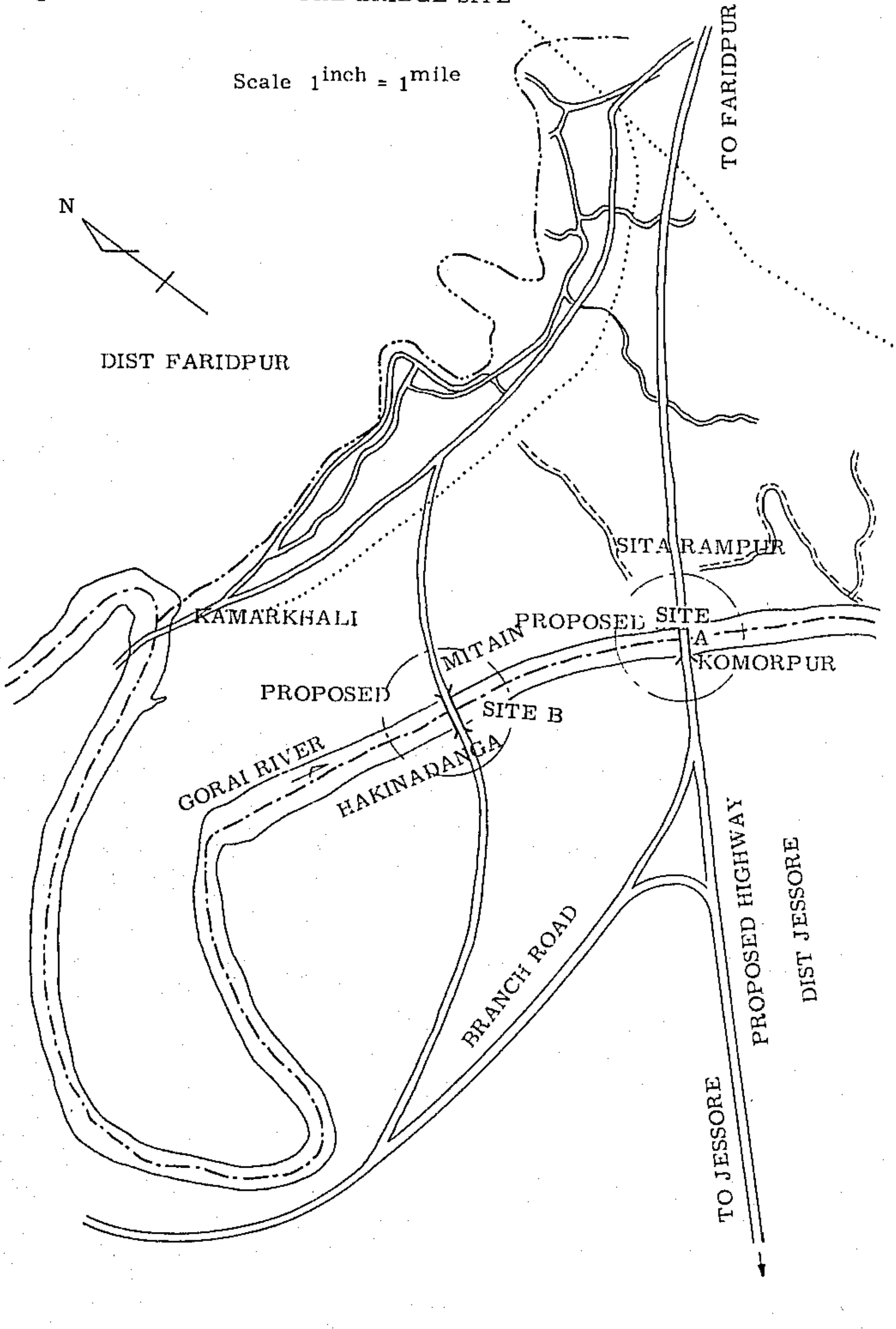
The Gorai River, the largest tributary of the Ganges, branches from its main stream at Kushita, runs through Kamarkhali and Bhatiafara, and reaches Gopalgang where it branches into a number of smaller rivers that flow into the Bay of Bengal.

It flows over a total length of about 350 km and has a surface slope of as little as 1/20,000 which is maintained even during flood season.

The three major rivers of East Pakistan, the Ganges, the Brahmaputra and the Meguna join at a point near Dacca, and branch into many tributaries flowing into the Bay of Bengal.

Of these three rivers, the Ganges is the largest, with a catchment area of approximately 900,000 km^2 (350,000 miles^2) at Haridinge Bridge upstream of a point of the tributary Gorai river and a total length of 2,600 km (1,600 miles).

Fig. 2 KEY MAP OF THE BRIDGE SITE



According to WAPDA's statistics, the highest discharge rate of the Ganges recorded in the past is 54,000 m^3/sec (1,900,000 cusec), and its theoretical flood discharge is estimated to be 75,000 m^3/sec (2,650,000 cusec).

Of the above-mentioned discharge of the Ganges, a maximum 3,300 m^3/sec (117,000 cusec), or 4,300 m^3/sec (150,000 cusec) in terms of theoretical flood discharge flows into the Gorai River.

Greater parts of East Pakistan are covered by alluvial soil, and deposit is still actively taking place at the river mouths.

In the flood season, as much as 40% of the entire area of East Pakistan is inundated. It attracted the attention of the Survey Mission that the districts extending along the Gorai River were also inundated during the flood season (only houses and roads showing themselves above water level). The flood takes place during the known period of the year when the water level rises up to almost the same level each year. Adaptability to this periodical flooding has therefore been attained for the cultivation of rice and jute which are the main agricultural products of East Pakistan.

(B) Water Level of the Gorai River

During the survey conducted in the rainy season of 1966, the water level of the Gorai River started to rise in early May, and after reaching the first peak on July 10, held at the same level until the beginning of August, then started rising gradually until September 2 when the highest water level was marked. Records for the past six years indicate that the highest level recorded is 29.15 ft which was observed on September 7, 1963. The highest level measured during the survey was only 0.06 ft below this record. The highest water level, which varies each year by a small margin, less than 1 ft (30 cm), takes place during the period from late August to early September.

Table 1 Water Level of the Gorai River during Recent Years

Year	Highest Water Level		Lowest Water Level	
	Water level (ft)	Date recorded	Water level (ft)	Date recorded
1961	28.96	August 29		
1962	29.10	September 6	7.80	May 1
1963	29.15	September 7	7.10	April 17
1964	29.10	September 13	9.95	April 7
1965	28.30	September 16	9.30	April 3
1966	29.09	September 2		

Note: Water level was measured at P.W.D. Datum level adopted was 1,509 ft lower than the mean sea-level of the Indian Ocean.

(C) Estimated High-Water Level

The High-water level of the Gorai River recorded at Kamrkhali is 28.0 — 29.0 ft and this value shows almost no fluctuation each year. Since no sufficient data was available to make probability calculations in respect to the estimated high water level of the Gorai, inference was made on the basis of results of probability calculation carried out for the city planning survey of Dacca.

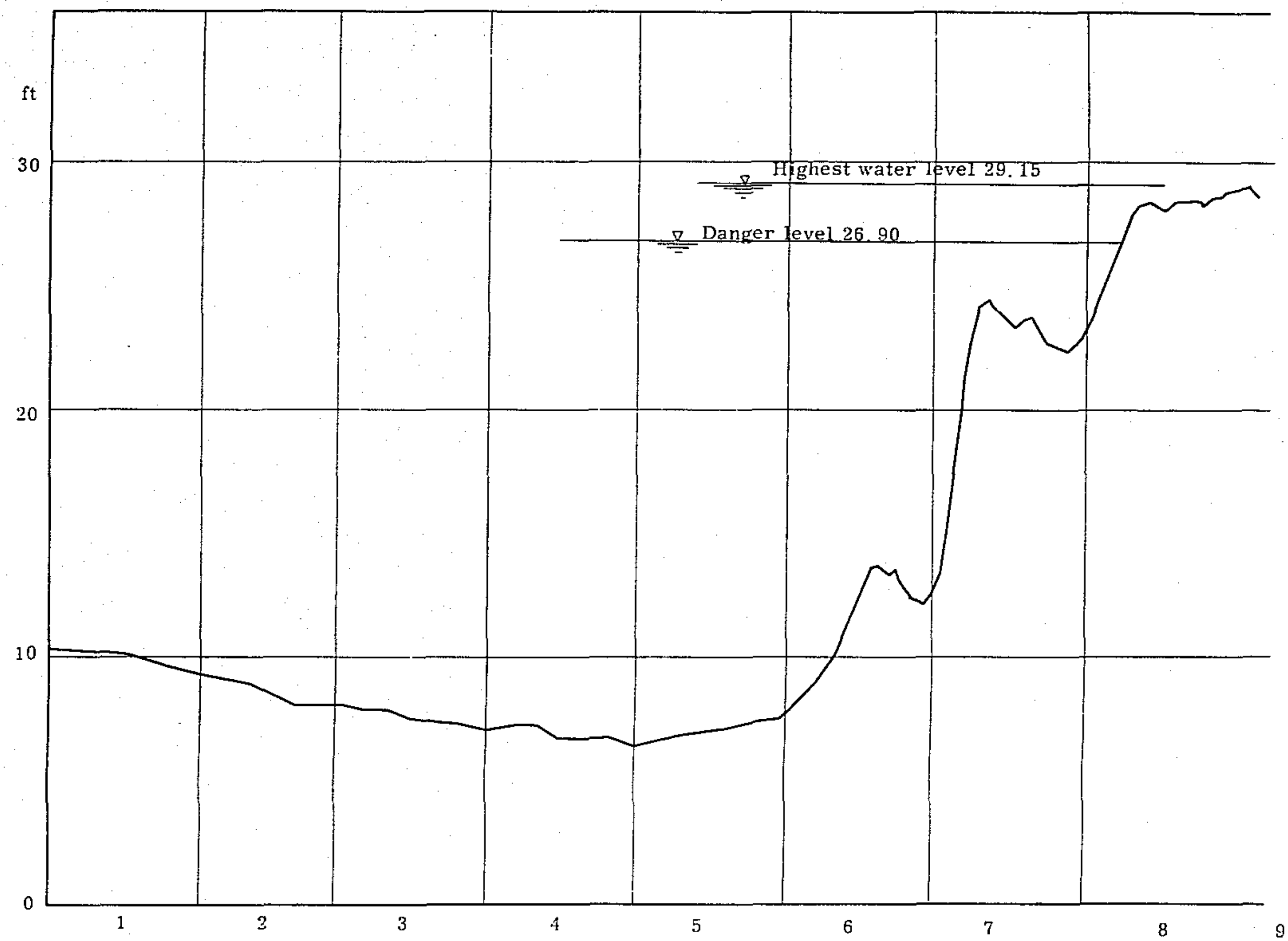
Estimated probability of high water level at Mill Barracks in 1955 is 23.1 ft for 1/80, 23.7 ft for 1/100 and about 25 ft for 1/700.

Since the highest water level recorded during the survey was 23.25 ft, it may be safely said that the probability that the water level rises by 2 ft is a remote possibility.

The above estimate is based on the assumption that the river would maintain the same condition in the future. If, therefore, any renovation or improvements made on the river, could result in a slight rise in water level.

Accordingly, the value measured during the survey plus 2 ft (0.60 cm) may be reasonably adopted as the estimated high water-level of the Gorai River for the designing of the bridge.

Fig. 3 Water level of Gorai River (At Kamarkhali) 1966



(D) Velocity and Discharge Rate during the Dry Season

Velocity measurement and sounding work carried out during the first phase survey revealed that the flow velocity of the river was 0.35 m/sec (1.13 ft/sec). This value was obtained by means of floating bamboo tubes measuring 8 cm (dia.) x 80 - 100 cm (length) which were arranged to float within a measuring distance of 200 m.

Since the cross-sectional area of the river at the time of this velocity measurement was approximately 3,120 m^2, the discharge rate, Q, can be expressed by the following equation.

$$Q = A.V = 3,120 \times 0.35 = 1,092 \ m^3/sec.$$

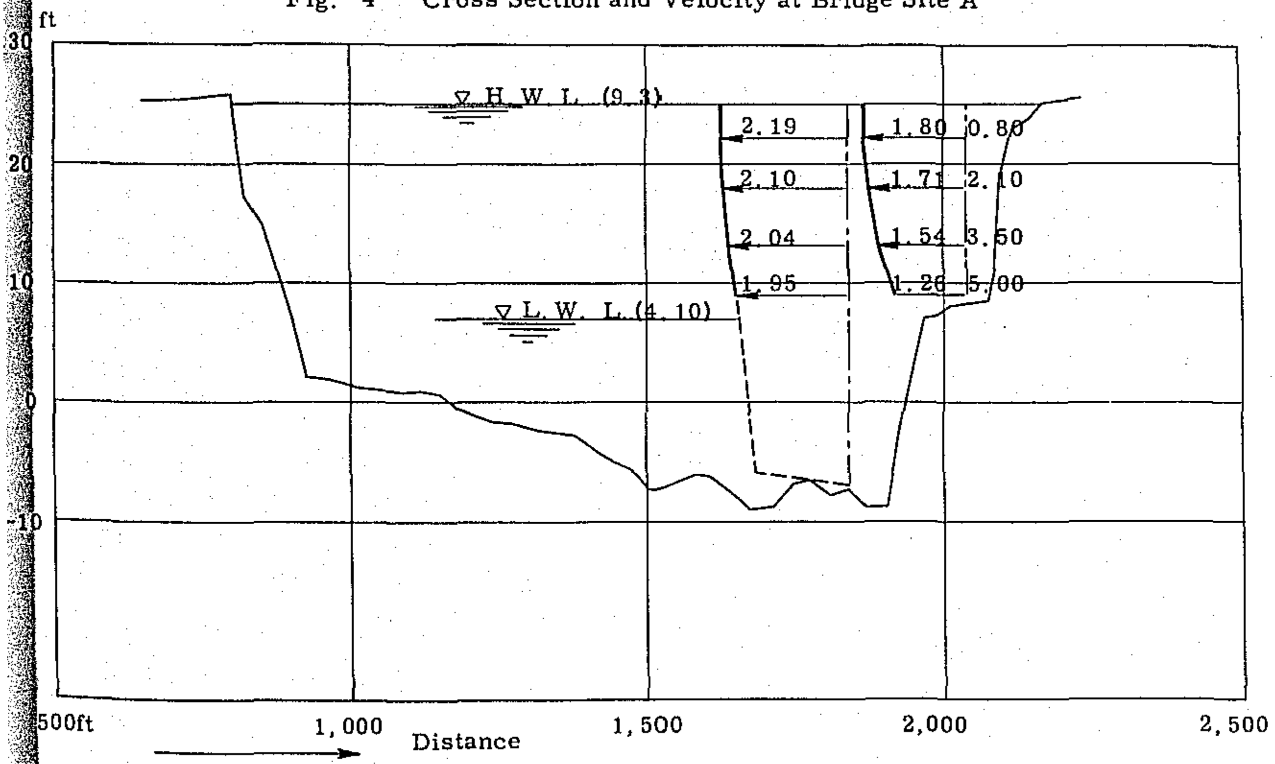
(E) Velocity and Discharge Rate during Flood Season

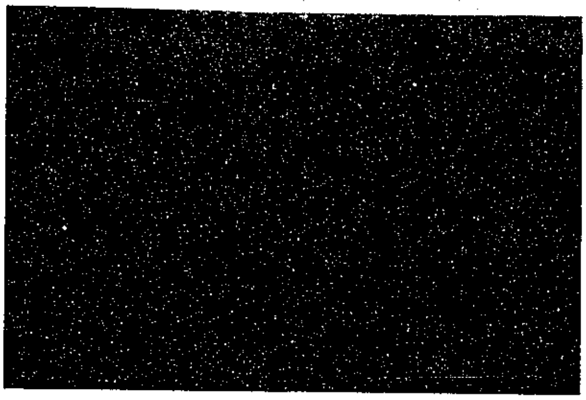
Measurement of discharge rate during flood season was carried out on September 3 when the water level marked a value close to the highest water level recorded in 1966.

Results of velocity measurement is as shown in Fig. 4

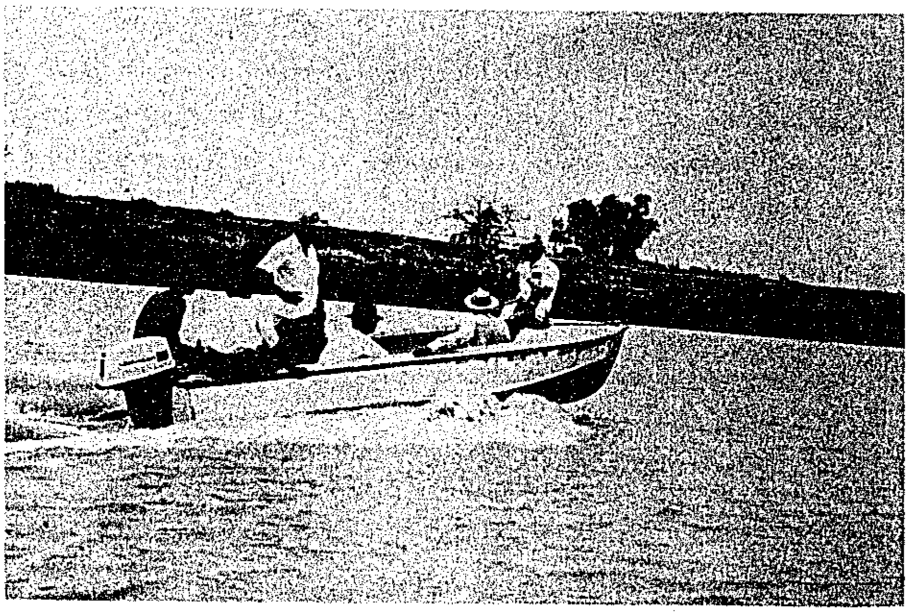
Price current-meters were used for velocity measurement which was conducted by dropping anchors to keep the boat at right positions since the meters could not have been fixed by any other means.

Fig. 4 Cross Section and Velocity at Bridge Site A





The Gorai River
in the flood season
(The second phase survey)



The river survey
by the speed boat

This, however, entailed some danger when the flow velocity exceeded 2 m/sec.

Although the observation was made only at two points, i.e., one with a comparatively shallow depth and another with great depth, the velocity obtained may be regarded fairly accurate since the measurement was conducted three times at each place by means of four reading points arranged perpendicular to the river bed.

The measurement revealed that the deep place marked 220 m/sec near the river surface, and the measurement was little influenced by the water depth, recording a velocity of about 2.00 m/sec at a depth of 5 m. At another place with a comparatively shallow depth located closer to the right bank, the velocity marked was 1.80 m/sec near the river surface and 1.3 m/sec near the river bed. It may be added that the maximum river velocity was measured near the river surface despite the wind blowing at a velocity of about 10 m/sec against the current during the work.

The mean velocity of the river as estimated from the above measurement was therefore concluded to range from 1.8 m/sec to 2.00 m/sec.

In obtaining the theoretical mean velocity by Manning's formula, the surface slope, I, may be expressed by the following equation since the distance and water level difference between Point A and Kamarkhali are 22 km and 1.20 m (approximate values) respectively.

$I \doteqdot 120/22.00 = 1/18,000$

And assuming that the roughness coefficient of water, n, is 0.020, the following equations may be established.

$V = 1/n \, R^{2/3} \, I^{1/2}$

where

$R \doteqdot 10$ m, $R^{2/3} = 4.6$, $I^{1/2} = 1/134$

Hence,

$V = 1/0.020 \times 4.6 \times 1/134 = 1.72$ m/sec.

It is quite conceivable that the roughness coefficient declines to less than 0.020 if a river with 10 m depth has a small flow velocity and a surface slope of about 1/20,000. The results of the above calculation are therefore considered to show a fair conformity to the measured value.

If the river bed is assumed to maintain the same condition as was observed by the survey in the dry season, the cross-sectional area would be

3, 120 m^2. The flood discharge during and based upon the velocity measurement may therefore be expressed in the following equations.

Velocity assumed to be 1. 80 m/sec:

$$Q = Av_1 = 3,120\ m^2 \times 1.80\ m/sec \doteqdot 5,600\ m^3/sec$$

Velocity assumed to be 2. 00 m/sec:

$$Q = Av_2 = 3,120\ m^2 \times 2.00\ m/sec \doteqdot 6,200\ m^3/sec$$

The above equations indicate that the flood was flowing at a discharge rate of about 6, 000 m^3/sec while the Mission was engaged in velocity measurement.

Data offered by E. P. WAPDA indicate that the highest flood discharge of the Gorai River, recorded in 1962, was 3, 300 m^3/sec, and that the theoretical flood discharge of the river is 4, 200 m^3/sec. While it is unknown on what basic calculations were made which produced these values, an outstanding discrepancy is noticeable between these values and those obtained by the actual measurement of the Mission. Although the river could not possibly have been widened rapidly during the past several years, it apparently tends to be gradually widening leading to the increasing discharge rate; and this, it appears, is well backed up by WAPDA's data which point out the frequent breakings of banks to which the river is subjected.

(F) Wave Height at Bridge Site

Although the weather was fair during the field survey during the rainy season, it was noticed that waves of the Gorai River were so high that it was felt dangerous to cross the river from the right to the left bank by boat. Wind velocity at that time is considered to have been about 10 m/sec, and it was blowing against the river flow. This wind was noted to be rather frequent during the rainy season, and the Mission was told that the wind velocity rises up to 45 m/sec (100 miles/hr) at times of rain-storms caused by cyclones.

On the assumption that the straight portion of the Gorai River is 20 km, and the river depth 10 m, the wave height may be obtained by Maliter's wave formula as expressed in the following equations and table.

$$H = 0.0612\sqrt{VG} + 0.762 - 0.27\sqrt[4]{F}$$

Wind velocity (V)	(m/sec)	10	15	20	30	40
Wave height (H)	(m)	1.06	1.25	1.41	1.69	1.93

These values conform nicely to the value of 1.0 - 1.5 m measured with the eye.

Application of Bietschneider's formula also leads to the wave height of 0.82 m and 1.35 m at wind velocity of 1 m/sec and 15 m/sec respectively.

Due to the lack of an actual measurement record, the maximum wave height at the bridge site to be taken into account for the design of the bridge could not have been obtained except by estimate. Considering such factors as the distance of the straight portion of river, river section and duration of wind, a maximum of 3.0 m may be considered ample.

3. Meteorological Survey

(A) Outline

Meteorological survey is indispensable for planning the execution of any construction work since it greatly affects the construction period, raising of necessary materials, and kind and arrangement of temporary structures and facilities.

The climate of East Pakistan can be broadly classified into two categories, the dry season which lasts for one half of the year from November to April and the rainy season which occupies the other half of the year from May to October during which 80% of the annual rainfall in concentrated.

Although the temperature is around 35°C, the humidity is extremely high, reaching 100% occasionally. At the beginning and end of the rainy season, rainstorms caused by the cyclones (correspond to typhoons that often pass through Japan) attack East Pakistan.

During winter (dry season) which follows the rainy season, fair and agreeable weather continues with low humidity and the temperature ranging between 30°C and 10°C.

(B) Temperature

Average temperature at Faridpur during the past several years, and average monthly temperature at Jessore, which were made available by the Highway Dept. of Eas Pakistan, are tabulated below.

Table 2 Average Temperature at Faridpur

Max. temperature (°F)		Min. temperature (°F)	
Max.	Average	Min.	Average.
100	75.9	46	52.5

Table 3 Average Monthly Temperature at Jessore (°F)

Month	Average Temp.	Month	Average Temp.
Jan.	64	Jul.	83
Feb.	69	Aug.	83
Mar.	78	Sep.	83
Apr.	85	Oct.	81
May	85	Nov.	73
Jun.	84	Dec.	68

(C) Rainfall

The pattern of rainfall during the rainy season in East Pakistan is more or less similar to that in Japan in respect to the annual rainfall and the rainfall per unit area and time. The difference is the concentration rainy days within the 6-months of the rainy season which results in more frequent rainfalls than in Japan.

The amount of rainfall has been long observed in East Pakistan by about 300 weather stations belonging to the Governments Meteorological Observatory. Amount of rainfall at Faridpur and Jessore obtained from the past record is tabulated below.

Table 4 Amount of Rainfall Observed at Faridpur Weather Station

1963	1964	Average annual rainfall	Max. monthly rainfall
57.31 in/yr	91.3 in/yr	66.7 in	51.2 in

Table 5 Amount of Rainfall Observed at Jessore Weather Station (In inches)

Month	Rainfall	Month	Rainfall
Jan.	0.4	Jul.	12.2
Feb.	1.3	Aug.	11.0
Mar.	1.9	Sep.	8.5
Apr.	3.5	Oct.	4.6
May	8.1	Nov.	0.9
Jun.	12.2	Dec.	0.2

(D) Humidity

Humidity observed at Faridpur Weather Station is as tabulated below.

Table 6 Monthly Averaged Humidity Observed at Faridpur Weather Station

Month	Jan.		Feb.		Mar.		Apr.		May		Jun.	
	M	E	M	E	M	E	M	E	M	E	M	E
Humidity	72	61	72	58	72	54	76	62	78	74	82	82
Month	Jul.		Aug.		Sep.		Oct.		Nov.		Dec.	
	M	E	M	E	M	E	M	E	M	E	M	E
Humidity	85	84	86	84	83	82	78	76	72	70	72	66

Note: M=Morning E=Evening

(E) Wind velocity

The maximum wind velocity observed in Faridpur and Jessore Districts is 100 miles/hr which is caused by rain-storms due to cyclones. Except the cyclone rain-storms, wind is rather light, but slightly more severe during the rainy season.

4. Soil Exploration

(A) Outline

Soil exploration was conducted by boring tests along Routes A and B which were selected for the proposed center line of the bridge for comparison between them in respect to soil formation and soil nature, whereby the foundation conditions of the two routes were examined to select the suitable routes. At first, the test borings were conducted at two points on each route. The test boring disclosed that Route B is provided with far better foundation conditions than are presented by Route A. Further borings were therefore carried at three points on Route B for examination of the soil formation as well as the soil nature which is required from the standpoint of foundation engineering.

Kind of work and equipment used are given below.

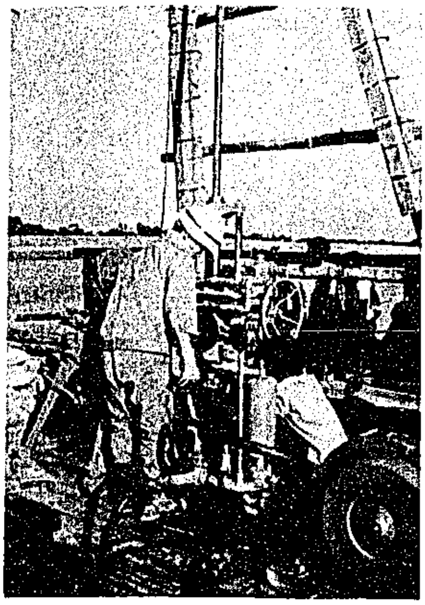
Kind of work:

1. Boring
2. Standard penetration test
3. Sampling of undisturbed soil
4. Laboratory test

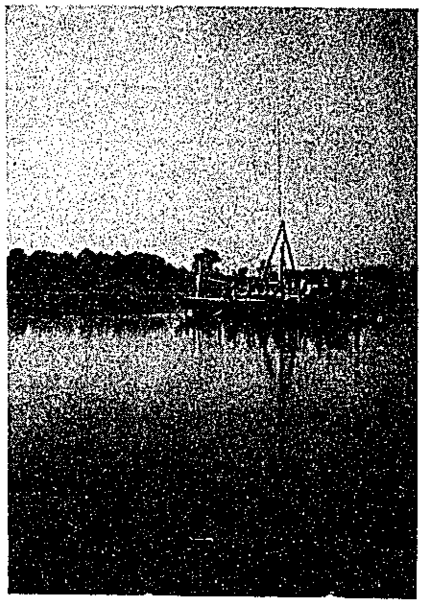
Equipment used:

1.	Tone's boring machine, Model UD-5	1 unit
2.	Kano's boring machine, Model KR-200	1 unit
3.	Marui's triaxial compression machine	1 unit
4.	Tanifuji's monoaxial compression machine	1 unit
5.	Soil testers	1 set
6.	Honda's generator	2 units

Figures relative to the soil exploration are shown in the table 7.



B3-point Boring
(The third phase
survey)



B3-point Boring,
scoffolding
(The third phase
survey)

The fixed piston type
sumpler with undisturbed
soil,B5-point
(The third phase survey)

Laboratory test
(The third phase survey)

Undisturbed soil,
B5-point
(The third phase survey

Table 7 Comprehensive Table of Volume of Works of Soil Exploration for Gorai River Bridge Construction Project, East Pakistan

Route	Item / Boring No.	Bore depth (m)	Standard penetration test	Sampling of undisturbed soil	Place of survey	Soil Exploration: Moisture content test	Specific gravity test	(A) (B)	(A) (C)	(D)	(E)	(F)
A	A-3	55.45	55	0	River	16	16	13	0	0	0	0
	A-4	60.55	59	0	"	13	13	0	0	0	0	0
B	B-1	50.19	50	0	Land	10	10	10	0	0	0	0
	B-2	45.22	45	0	River	11	11	11	0	0	0	0
	B-3	62.00	63	0	"	21	21	21	0	0	0	0
	B-4	50.16	50	0	"	15	15	15	0	0	0	0
	B-5	45.70	37	7	Land	9	9	9	7	7	7	7
Total		369.27	359	7		95	95	79	7	7	7	7

Note:
(A) - Grading
(B) - Sieve analysis
(C) - Hydrometer Test
(D) - Liquid and plastic limit test
(E) - Monoaxial compression test
(F) - Triaxial compression test

(B) Method and Purpose of Soil Exploration

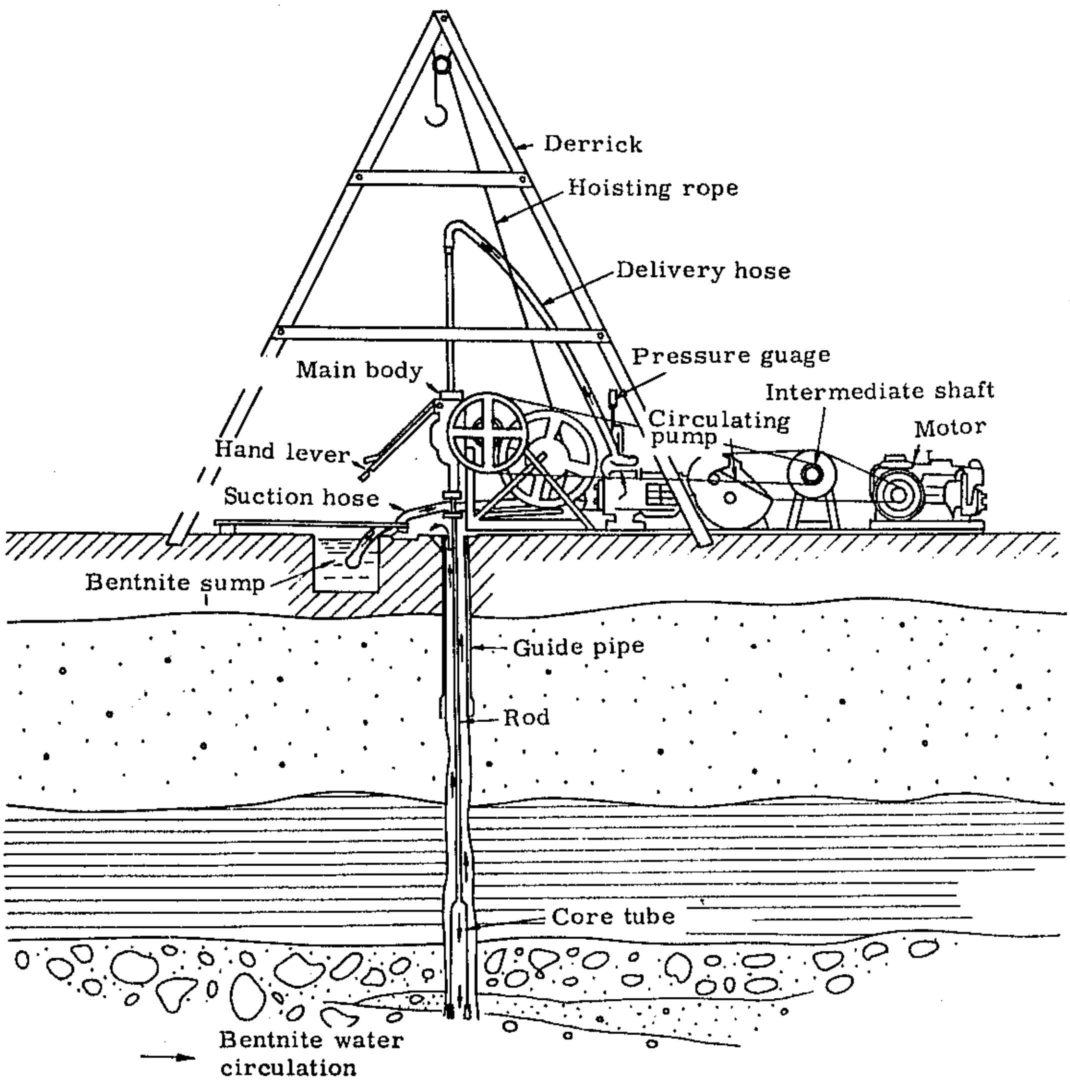
(1) Boring machines

Equipment used was Tone's boring machine, Model UD-5 (1 unit) and Kano's boring machine, Model KR-200 (1 unit). The former is a hydraulically operated unit, while the latter is manually operated utilizing the principles of the lever and fulcrum, and their nominal capacity is 150 - 200 m.

Boring was carried out by the washing method in which bentnite-water was pumped into the core tube or cross bit (outside diameter: 85 mm) for protecting the boring hole.

Observation and discrimination of the formation was carried out based on the test pieces subjected to standard penetration test conducted at each 1 m depth, and on undisturbed samples collected.

Fig. 5 Common Boring Machine Use



(2) Scoffolding

For the borings conducted at five points on the river out of the total seven boring sites, scoffolding equipped with the frame were used. Each scoffolding was made up or two ferry boats closely tied with each other and fixed at the specified position on the river by casting buch anchors. The ferry boats thus tied up had sufficient stability and were spacious enough to accomodate 20 persons with an area of 9 m^2.

(3) Standard penetration test

The standard penetration test was performed according to the method specified in JIS-A-1219, as briefly explained blow.

A rod of 4.15 cm long was suspended through the Raymond sampler measuring 81 cm (total length) x 5.1 cm (outside dia.) x 3.5 cm (inside dia.) which was arranged through the excavated hole as far downwards as the bottom. A hammer weighing 63.5 kg was suspended from 75 cm above the fixed point and dropped to give percussion to the layer, whereby the compactness and strength of the layer at its original position was measured by the number of percussions required for the sampler to penetrate each 30 cm (Value of N). Test Pieces obtained by this penetration test were sealed in polyvinyl bags for laboratory tests.

(4) Sampling of undisturbed soil

Undisturbed soil was collected by means of a sampler with the fixed piston which was driven in hydraulically with care.

The sampler, having the structure shown in Fig. 6, has the following features.

(a) When the sampler is driven in at a static pressure, the fixed piston serves to cause no load (i.e., piston weight, pressure of muddy water) on the soil specimens, which makes the sampling of undisturbed soil possible.

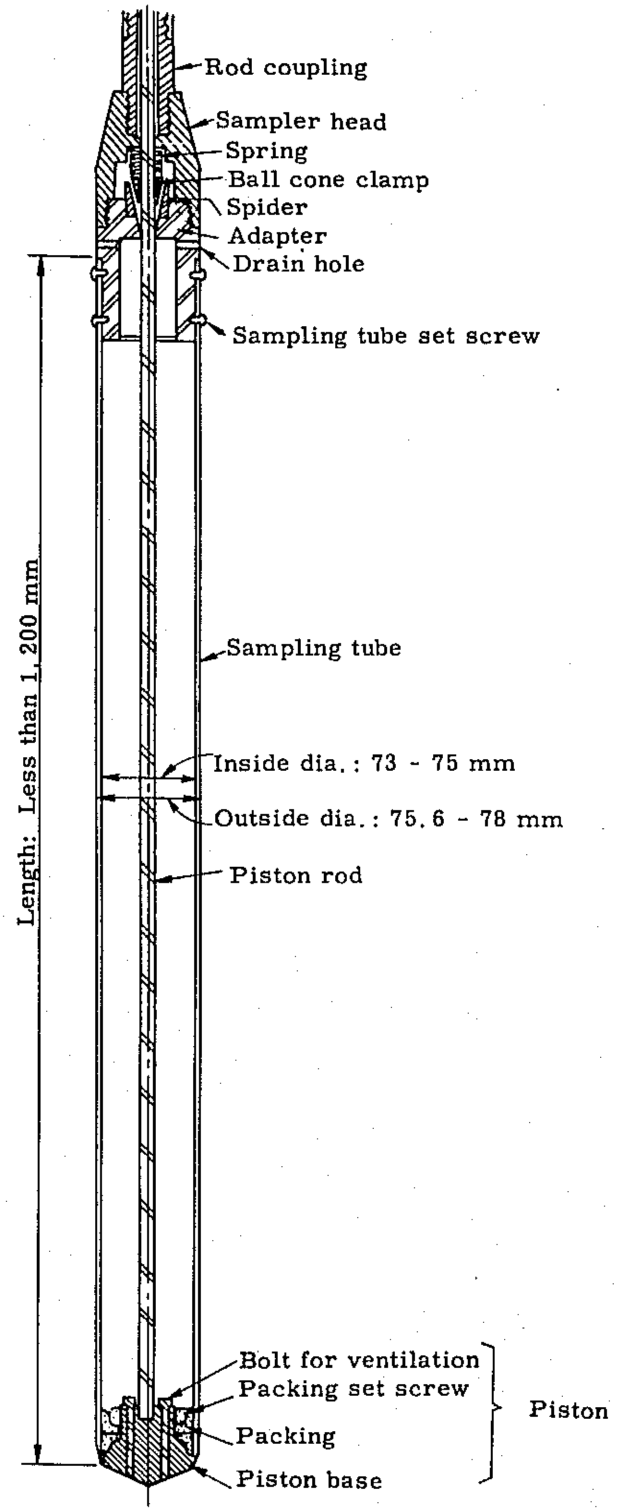
(b) 100% sampling ratio can be attained.

(c) Sampling ratio can be detected accurately.

These excellent features, available only with the above-mentioned fixed-piston type sampler, are best fit for sampling soft clay and silty soil. The brass-made thin wall tube has the following specifications.

Diameter	:	73 mm
Wall thickness	:	1.2 mm
Ratio of inside dia.	:	1%
Angle of cutting edge	:	6%
Length	:	110 cm

Fig. 6 Example of Fixed-Piston Type Sampler



(5) Laboratory test

Laboratory test was carried out in accordance with the method specified in JIS (Japanese Industrial Standard). For test items not specified in JIS, methods adopted by the Japan Soil Inst. were applied.

(a) Specific gravity test of soil grain (conducted according to the method specified in JIS-A-1202)

This test was conducted by means of a pycnometer to obtain the true specific gravity from the relationship between the volume and weight of soil. The purpose of this test is to determine the components of soil and to obtain physical characteristics such as grain size, etc.

(b) Moisture content test (conducted according to the method specified in JIS-A-1203)

The soil was dried at 110°C ± 5°C in a drying chamber for 24 hours to measure the moisture content of dried soil. This test was conducted mainly to obtain the soil nature and other physical characteristics of soil by calculation.

(c) Mechanical analysis of soil (conducted according to methods specified in JIS-A-1204)

For sandy soil, sieve analysis was conducted, whereas precipitation analysis utilizing hydrometer was carried out for clayey soil. The purpose of this test is to classify soils by grain size and to obtain physical characteristics of soil such as permeability coefficient, etc.

(d) Liquid limit test of soil (conducted according to method specified in JIS-A-1205)

(e) Plastic limit test of soil (conducted according to method specified in JIS-A-1206)

This test was performed to obtain the plasticity of soil for classification of soil nature. The test, which also can clarify the stability of soil, was carried out by means of a liquid-limit device, and the moisture content measured when 25 drop impacts were given was taken as the liquid limit. The plastic limit was obtained by shaping the soil specimen into a stick-like shape (3 mm dia. approx.) while dehydrating it, and by measuring its water content immediately before it crumbled.

(f) Monoaxial compression test of soil (conducted according to method specified in JIS-A-1216)

This test was performed on test specimen measuring 3.5 cm (dia.) x 8.0 cm (height) to obtain the strength and sensitivity of soil by means of the manually operated Tanifuji's monoaxial compression tester. Results of this test serve the purpose of estimating the bearing capacity of soil.

(g) Triaxial compression test of soil (conducted according to methods adopted by the Japan Soil Inst.)

This test was conducted in the same way as applied in the above-mentioned monoaxial compression test, but with glycerine solution used to give lateral pressure on the test specimen. The lateral pressure was varied over several stages to obtain the cohesion and internal friction angle of soil, thereby to analyze the bearing capacity. The equipment used was the Marui rotorized traxial compression tester.

(C) Exploration Results

(1) The two sites selected for the proposed bridge site, situated about 10 km and 13 km respectively downstream of Kamarkhali, are included, topographically, in the extensive area covered with alluvial soil. Meanders common to tributaries of the Ganges are most prominent at Kamarkhali in the case of the Gorai River. Discharge rate and velocity of the river rise substantially in the rainy season. In the dry season, however, both the velocity and water level decline drastically, and the river bed presents an irregular surface which allows shoals to form. This irregularity of the river bed does not remain the same, but changes each year, which is indicative of the severe erosion and heavy transportation and deposition of sand and silt to which the river is constantly subjected. The soil of the river bed and banks is composed predominantly of fine sand, yet no measures are seen to be taken for bank protection. The river is therefore in an extremely dangerous natural condition that allows the floods to make the river section become gradually wider and the depth eventually to become deeper at some sections, which can only result in more aggravated meanders.

(2) Water level and conditions of river bed.

Since the survey was conducted during the dry season, from January to March, a remarkable decline of water level was noticed. The water level obtained by the leveling work carried out by theMission was 5.4 m lower than the level of the head of pile No. 160 driven on the Faridpur side, and shoals of comparatively large size, which had not been seen the previous year, were found formed closer to the Faridpur side. The river bed irregularity in the flowing direction of the river was also noted conspicuous, and this was considered to certainly lead to the constant shifting of the centre line of the stream.

Table 8 shows the river bed level at points where the boring was performed.

Table 8 River Bed Level and Water Depth

Boring No.	B1	B2	B3	B4	B5
Level (m)	+7.62	+2.70	-0.94	-0.38	+7.79
Depth (m)	0 (Boring site on left bank)	0 (Boring site on right shoal	1.26	2.58	0 (Boring site on right bank)

(3) Soil formation

The soil formation, shown in the separate Fig. 19, "Estimated Soil Profile," and in the attached Tabel-9, can be outlined as follows.

Soils on both route A and B are predominantly composed of fine sand and medium sand. The clay soil layers which lie between sand layers were discovered at two boring sites. The clay soil also covers the ground surface of both banks. One of the two clay layers was about 1.5 m thick as discovered by the boring at Site B-5 on the right bank (Jessore side) and the other was confirmed to be less than 1 m in thickness by the boring at Site A-4. The surface soil covering both banks is the silt layer having a thickness of 2 - 2.5 m. It is quite likely, however, that layers of clay soil are deposited on Jessore side of Route A as is the case with Route B.

Sand layers are generally formed by alternating layers of fine sand and medium sand on both routes. It was noted, however, that the foundation layer, which is most important, was found deep on Route A by the comparative boring performed at two points on either route. Therefore, route B was chosen for the proposed site of the Gorai River Bridge, and the outline of layers given below deals only with Route B.

(1) Surface layer

The surface layer, composed of silty soil with covers both banks and extends 2 - 2.5 m downwards from the ground surface, is utilized at present for the cultivation of agricultural products. This layer is generally brownish in colour

(2) Upper sand layer

Composed of alternating layers of fine sand and medium sand, and presenting brownish or dark greyish-blue colour, this layer has a relatively small compactness with its N-value registering less than 20. Its vertical extension ranges between 8 and 12 m. On the right bank, however, its thickness decreases down to 3 m., which is ascribable to the fact that clay soil layers found on the right bank with a thickness of as large as 15 m are most heavily eroded on the middle of the river bed.

(3) Clay soil layer

As already mentioned, the clay soil layer is found only on the Jessore side. With a thickness of 15 m, it can be subdivided into upper and lower layers by the difference in relative density.

(a) Upper clay layer

This layer generally is dark greyishblue in colour, with some parts tinted with yellow. Its N-value ranges between 10 and 13, indicating that consolidation has substantially developed. This layer lies in the upper part of the clay soil layer, and has a thickness of about 7 m.

(b) Lower clay layer

This layer is dark grey in colour, and has a relative density smaller than that of the upper clay layer.

Its N-value is not clearly known since sampling was made during the continuous penetration test. However, judging from the pressure required for penetrating the sampler and the strength obtained by the monoaxial compression test, it may reasonably be assumed that the N-value ranges between 6 and 9. This layer differs from the upper clay layer in that it includes corroded plants in their original form.

(4) Intermediate sand layer

The sand layer with N-value exceeding 20 was classified as the intermediate sand layer. This layer is brownish as well as greyish in colour as in the case of the upper sand layer. Its thickness, which is quite large ranging from 26 - 30 m. tend to decrease in the direction of the Jessore side where it marks only 8 m. thickness. This is indicative of the shallow supporting layer on the Jessore side.

(5) Lower sand layer

In the "Estimated Soil Profile," the upper limit plane of this layer is shown by bold line, and this plane is considered as the supporting layer with N-value exceeding 50. The level of the supporting layer which is 34 m at Faridpur side tends to decrease in the direction of Jessore side, i.e., the level at the middle of the river bed is approximately 30 m and that on the bank of Jessore side about 21 m.

When compared with the upper and intermediate sand layers, this lower sand layer is characteristic in that it contains little mica and is composed of two layers presenting different colours, i.e., greyish blue and greyish green.

One of the causes of this difference in colour, it may be deduced, is that the two layers were created by different processes of soil formation.

The explanations given above are briefed in Table 9 Stratigraphic Structure. Distinction between alluvial soil and diluvial soil cannot be made until fossil plants and animals included in the soil are examined. Judging from the soil nature, however, it may reasonably be assumed that alluvial and diluvial layers are devided by the upper limit plane of the lower sand layer.

Table 9 - Stratigraphic Structure

Strata	Layer	Thickness (m)	Colour	Characteristics
(1)	Surface layer	2.0 - 2.5	Brown	Silty soil covers both banks
(2)	Upper sand layer	3 - 12	Greyish brown or Dark Greyish	Thickness ranges between 8 - 12 m except on the Jessore side where it marks 3 m only
(3)	Upper clay layer	7	Yellowish brown	Substantially compact with N-value registering 10 - 13. Fossil plants are included in a limited quantity.
(4)	Lower clay layer	8	Dark greyish blue	A large quantity of fossil plants retaining original shape are included. Less compact than the upper clay layer, with a slightly smaller relative density.
(5)	Intermediate sand layer	8 - 30	Greyish brown and Dark greyish blue	Composed of alternating layers of fine sand and medium sand as in the upper sand layer. Thickness declines to 8 m on the Jessore side.
(6)	Lower sand layer		Dark greyish blue and Greyish green	Composed of extremely consolidated fine and medium sand, with less mica inclusion than the overlying sand layers. Presents two colours, i.e., dark greyish blue and greyish green.

(D) Soil Nature

Given below is the soil nature of main layers obtained by the standard penetration test and physical test conducted on sandy soil as well as by physical and mechanical tests carried out on undisturbed soil specimens collected from clay layers.

(1) Standard penetration test

Results of this test are as tabulated below.

Table 10 - Layers and N-Values

Strata	Layer	N-Value	Bearing Capacity
(1)	Surface layer	Less-than 10	Poor
(2)	Upper sand layer	Less than 20	Poor
(3)	Upper clay l ayer	10-13	Good
(4)	Lower clay layer	(6 - 9)	Poor
(5)	Intermediate sand layer	20 - 50	Good
(6)	Lower sand layer	More than 50	Excellent

Note: Values in parentheses are estimated values

As indicated in the above table, N-value of more than 50 is noted partially even in the intermediate sand layer. Generally speaking, however, layers overlying the intermediate layer are inferior in bearing capacity. The sandy foundation found in the lower part of the lower sand layer whose upper limit level is 30 - 21 m is therefore the best supporting layer.

N-Values of sandy soil in relation to the relative density are indicated in Table 11 below.

The table clearly indicates that the compacted lower sand layer should be chosen for supporting the foundations of heavy structures.

Table 11 N-Value and Relative Density of Sandy Soil

N-Value	Relative Density
0 - 4	Extremely Loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Compact
More tahn 50	Extremely compact

N-value - relative density relationship for clay soil can also be indicated in a similar way as tabulated below.

Table 12 - N-Value and Relative Density of Clay Soil

N-Value	Relative Density
0 - 2	Extremely soft
2 - 4	Soft
4 - 8	Medium
8 - 15	Consolidated
15 - 30	Very consolidated
More than 30	Extremely consolidated

(2) Results of soil exploration

Results of soil exploration, given in detail in the attached comprehensive table and Report on Test Boring, may be summarized as follows.

(a) Physical characteristics of sandy soil

A characteristic common to all sandy layers is that they have an excellent siftable nature. The grain-size accumulation curve shows a high gradient, with the uniformity coefficient marking a value of less than 3 which indicates that the sandy soil is composed of grains of fairly uniform size. The curve also serves for distinction between fine sand and medium sand. Formation of such sand layers is obviously attributed to the topography and climate. To be precise, the easy river bed slope which extends over a long distance has served for developing siftable nature while the seasonal difference in flow velocity and discharge rate has created these sandy layers. The specific gravity measured ranges mostly from 2.6 to 2.7, although a small value less than 2.5 was registered in some parts indicating susceptibility to erosion and soil transportation. Clay content in sandy layers mostly registered 5 - 20 %, while exceptionally large values ranging between 20 and 30% were occasionally seen to indicate the inclusion of layers of small grain size.

(b) Physical characteristics of clay soil

The natural water content is small in the upper clay layer, marking wn = 34.5 - 52.8. The relationship between this natural water content and the liquid limit is, as shown in Fig. 7, expressed by the inequality, $Wn < WL$, which, coupled with the large flow gradient that ranges between 8 and 15, makes it a remote possibility that this layer may at any future date be subjected to liquifaction by the change in the impulsive water content. Casa Grande's plasticity indicated in Fig. 8 is ML in the upper clay layer, CH or Oh in the lower layers. Consistency is higher in lower layers than in the upper layer. The specific gravity of the lower clay layer is 2.5, indicating that it is organic in nature.

(c) Mechanical characteristics of clayey soil

Results of mechanical test conducted by means of monoaxial and triaxial compression tests are tabulated below for comparison between the upper and lower clay layers.

Table 13 - Results of Monoaxial Compression Test

Name of Layer	Compressive strength	Compressive strain	Sensitivity ratio
Upper clayey layer	1.63 - 2.31 qu (kg/cm²)	6.1 - 11%	1.8 - 4.4 Sr
Lower clayey layer	0.35 - 1.57 qu	15	2.9 - 6.2 Sr

The lower clay layer, which has less strength than the upper layer, has a sensitivity ratio exceeding that of the upper layer and is therefore vulnerable to strength decline once disturbed. Although a lower layer usually has larger strength than an upper layer overlying it, the compression test revealed that quite an opposite condition exists with this lower layer, and this has attracted the Mission's attention. It was noted, however, that its strength tends to grow larger at deeper points.

Table 14 - Results of Triaxial Compression Test

Layer name	Cohesion C (kg/cm^2)	Internal friction angle (φ°)
Upper clay layer	0.70 - 0.78	13 - 17
Lower clay layer	0.32 - 0.58	15 - 17

Since the triaxial compression test was carried out by the non-draining method without considering the increase in the shearing strength by consolidation. the internal friction angle is assumed to actually surpass the values given in the above table. In designing, however, it would naturally be advisable to adopt these values for safety's sake.

5. *Ground Water level*

During the boring carried out on both banks on Route B (Boring Site B-1 and B-5), the water level in the excavated holes were measured. On the left bank (Faridpur side), the ground water level was found to be GL-52 (EL 2.4 m) which is equivalent to the level of the river. On the right bank, (Jessore side), the values measured marked GL-3.6 (EL 3.2 m) which is about 1 m higher than the river level. This discrepancy is considered attributable to the difference in layers constituting the two banks, i.e., while the layers with good permeability cover the left bank, the clay layer with a thickness of 15 m is deposited on the right bank.

The water levels in the excavated holes do not necessarily conform to the true ground water level. It is therefore recommended to adopt the level of the river as the design ground water level.

Fig. 7 - Relationship between Natural Water Content(W) and Liquid Limit (WL)

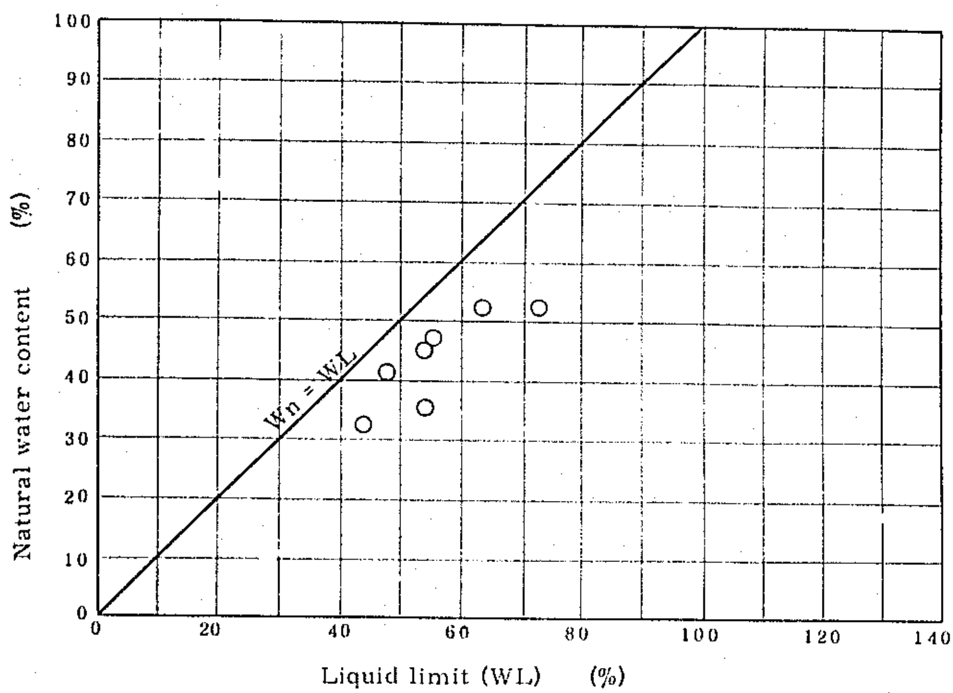


Fig. 8 Relationship between Plasticity Index (IP) and Liquid Limit (WL)

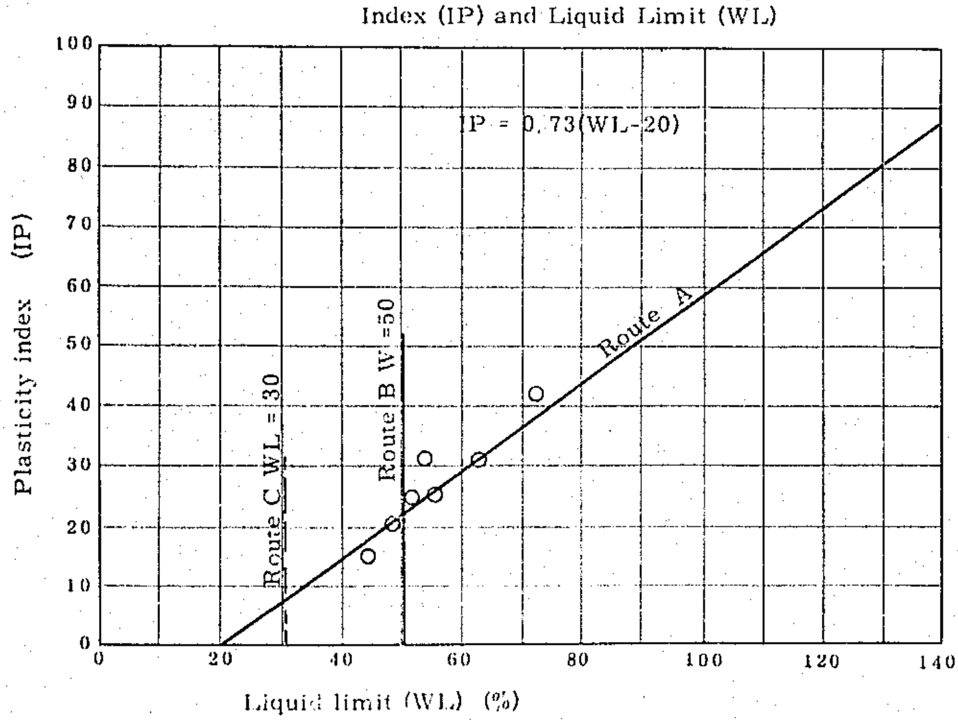
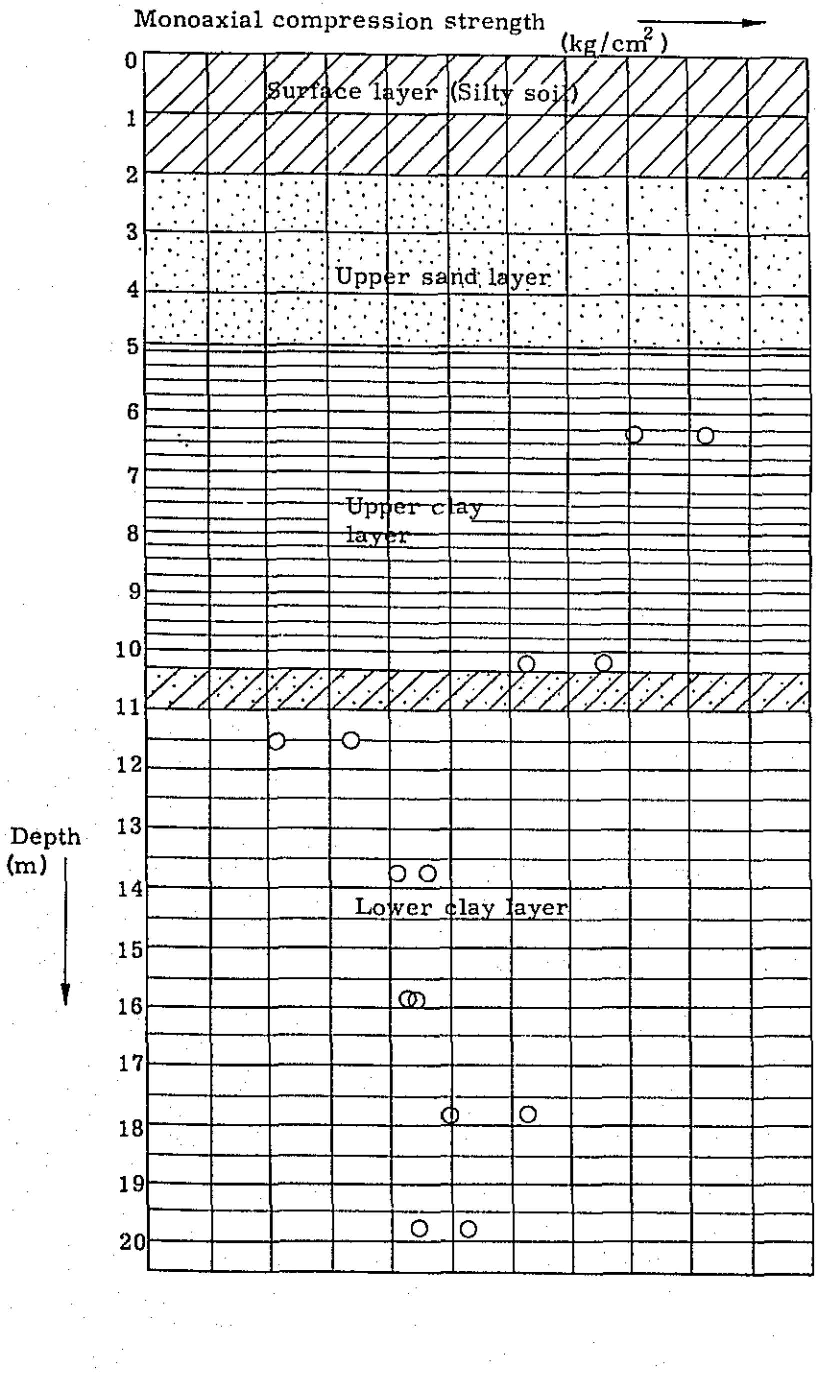


Fig. 9 Relationship between Monoaxial Compression Strength and Depth of Layer



CHAPTER III TECHNICAL CONSIDERATIONS

CHAPTER IV OUTLINE OF PRELIMINARY DESIGN

1. Suparstructure of P.C. Bridge

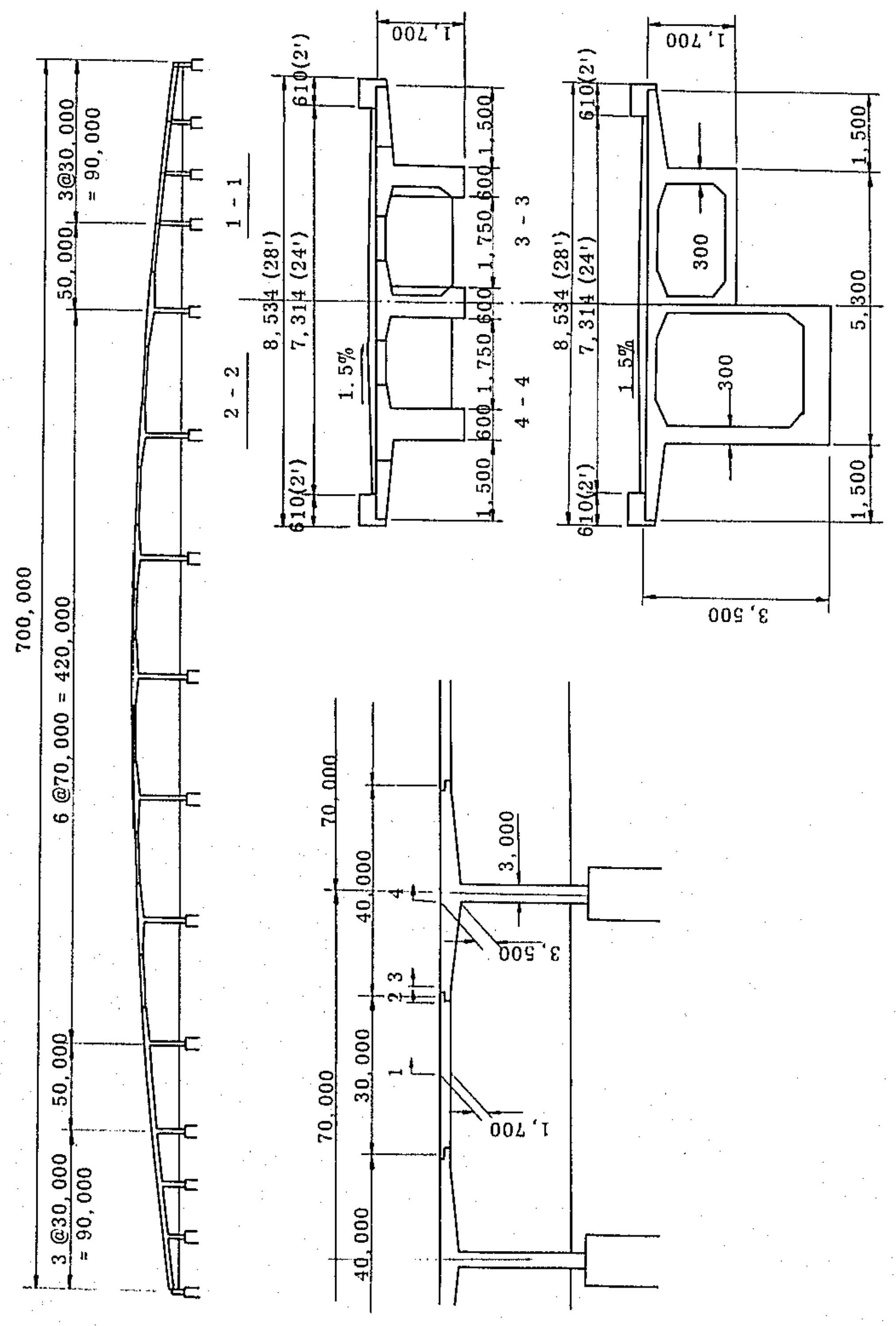
(A) Conditions for Design

Type	: Gerber overhanging girder (Dywidag type P.C. Bos Girder)
	: Suspension girder, Simple girder (Freyssinet type P.C. T-shaped girder)
Length of Bridge	: 700.000 m
Span Arrangement	: 3 @ 30.000 m + 50.000 m + 6 @ 60.000 + 50.000 m + 3 @ 30.000 m
Effective width	: 7.315 m
Load	: H-20
Impact Coefficient	: i = 50/I + 125 L ; ft.
Longitudinal slope	: 4.0 %
Breaking Safety	: $M_B / 1.75 (M_D + M_L) > 1.0$
Lateral Grade	: 1.5% para
Seismic coefficient (horizontal)	: Kh = 0.05
Temperature change	: ±20° C

Material Strength and Allowable Stress

(1) Concrete

Design strength (age=28 days)	
At time of prestressing -	: σ 28=350 kg/cm^2
Gerber overhanging girder	: σ ci=200 kg/cm^2
Suspension girder, Simple girder	: σ ci=300 kg/cm^2
Immediately after prestressing (compression)	: σ cat=150 kg/cm^2
" " (Tension)	: σ cat^1=-13.5 kg/cm^2
At time of imposing design load (compression)	: σ ca=115kg/cm^2
" " (Tension)	: σ ca^1 =-13.5 kg/cm^2
" " (Diagonal tensile)	: σ Ia = -18 kg/cm^2



700,000
3@30,000 = 90,000
50,000
6 @70,000 = 420,000
50,000
3 @30,000 = 90,000
1 - 1
2 - 2
3 - 3
4 - 4
8,534 (28')
7,314 (24')
610(2')
1,700
1,500
600
1,750
1.5%
300
5,300
3,500
70,000
40,000
30,000
3,000

Diagonal tensile at time of imposing breaking load (allowable)	: σ_{Ia} = -18 kg/cm^2
" " (Maximum)	: σ_{Ia} = - 36 kg/cm^2
Concrete joints of suspension girder and simple girder	: σ 28 = 300 kg/cm^2

(2) P.C. Steel cable (ϕ7)

Tensile strength	: σ pu = 155 kg/cm^2
Yield point stress	: σ py = 135 kg/cm^2
Allowable tensile stress (at time of desingning)	: σ pa = 93 kg/cm^2
" " " (at time of cable tightening)	: σ pi = 121.5 kg/cm^2

(3) P.C. Steel wire (ϕ27) SBPC 105

Tensile strength	: σ pu = 105 kg/cm^2
Yield point stress	: σ py = 60 kg/cm^2
Allowable tensile stress (at time of designing)	: σ pu = 60 kg/cm^2
(at time of cable tightening)	: σ pi = 72 kg/cm^2

(B) Outline of Structure

(1) Overhanging Girder

The overhanging girder has the statically determinate structure with suspended girder positioned on the center of span.

One box section suffices to cover the design effective width of 7.314 m but in this case 2-box type was adopted since it was considered preferable to make the box girder web fit with the suspension girder at the Gerber hinge part.

The dimension of the web was designed to be 30 ^{cm} which is a value required for anchoring P.C. steel cable.
The web is not to be widened always through the girder.

The upper floor slab is not to be tightened laterally since the webs are arranged in 2.5 m, and is to be made of reinforced concrete.

(2) The piers are designed to create no bending moment by the axial force alone when the static load is imposed or live load is symmetrically imposed. However, to cope with a large bending moment which is created when the live load is imposed on one side of the span, so that made hollow, with their cross-section being oval shaped, so that sufficient rigidity may be obtained.

The pier top is the rigid joint at which the main girder and the pier are connected and which serves to transmit the bending moment from the main girder to the pier.

Of the two types of the pier, i.e., triangular truss type and common pier type to connect to the main girder, the latter was preferred since the pier is made of reinforced concrete.

(3) Suspension Girder and Simple Girder

As regards T-shaped simple beam, the number of main beam was limited to three to save the cost of form work and erection. The outside beam was made to have a non-uniform section so that it may match the overhanging box girder for the sake of beautility.

The inclination of the outsidebeam against the principal axis is adjusted by the upper floor slab and reduced to approximately 0° (0°-10')

(4) Bearing Shoe

The bearing shoe must permit the movement in the axial direction of the bridge since the difference in deflection is caused by live load on either end of suspension girder because of the bridge being Gerber type.

The shoe to be used for this purpose is the one which sandwiches the friction steel plate in neoprene, so that the sliding movement may be supported by the friction steel plate, and Neopreme takes the rotational force.

(5) Erection

In the case of the proposed Gorai River Bridge which is long and whose construction can be carried out only within a certain period of the year, it is most comendable to adopt the cantilever method in which the construction is continuously performed from the pier.

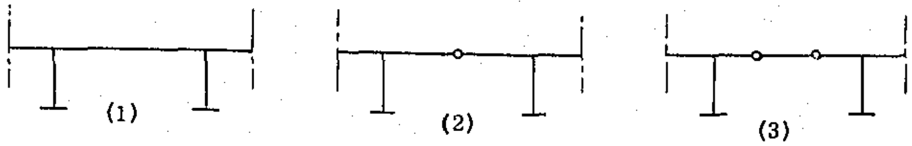
The cantilever erection method can be divided into two kinds. One of them is to apply cast-in-place concrete for the girder, and the other is the jointing of precast concrete blocks.

Utilization of precast concrete blocks has many advantages, but if the bridge is long, both the height of girder and the weight per block become inevitably large, and so inviting the necessity for a large amount of equipment and greater techniques.

Since the height of girders varies in the proposed bridge, a good number of non-interchangeable forms are required. Cantilever erection by the Dywidag method has therefore been adopted.

(6) Mechanical characteristics

A large bending moment is created at a point near the supporting point if the bridge has long spans and is provided with the most desirable mechanical characteristics. In the mechanical system of such a bridge, large crosssections are required near the supporting point to resist the bending moment and increased dead load, but the increase in cross-section and dead load gives extremely little influence on the increase of the bending moment.



(1) (2) (3)

The following illustrates the mechanical characteristics of three different types of intermediate spans.

Type (1), which does not have expension joints and has excellent road utility is subject to the development of large stress by temperature change if such arrangement is applied over many spans. Erection of a bridge of this type should be carried out by the method which makes the bending moment during erection close to the value created when all the girders were erected on the timbeiing.
Also, since the maximum positive bending moment occurs near the center of the span, larger cross-section is required, giving adverse effects with the dead load moment.

Type (1) is therefore less advisable than Types (2) and (3).

Type (2) has the rigid frame structure provided with hinges which transmit only vertical load at the center of span and do not prevent the horizontal movement.
If the bridge of this type is built by cantilever erection method, prestressing at the time of design load imposition would ensur safety of work execution.

Also, since P.C. steel cables are to be provided in a large quantity only near the supporting point developed by the maximum bending moment, i.e., the amount of steel cables to be arranged near the end, where the bending moment is practically c, is negligibly small, this type of span may be considered most economical for P.C. bridges.

The only drawback of this type of bridge is that the deformation of hinges provided at the centre of spans becomes large. In the United States and France, this hinge deformation is not considered acceptable, while in Germany, no particular attention is paid to the deformation but efforts are exerted for reinforcing the expansion joint part.

In Type (3), Gerber girder is positioned at the centre of the span, and no hinge deformation occurs. This type is inferior to type (2) in that the vorbauwagen for cantilever erection cannot be used as far as the centre of the span.

When compared with type (2), type(3) has more expansion joints, and Gerber hinges are apt to be soiled by rain water, etc.
However, since it is completely free from the hinge deformation, this type is considered most expedient for the proposed bridge.

2. Superstructure of Steel Bridge

(A) Conditions for Design

For the designing of the superstructure of steel bridge, comparative study was made on the following three specifications.

Specifications for Designing Steel Highway Bridge
(Japan Highway Association)
Highway Bridge Code for East Pakistan, 1962
Standard Specifications for Highway Bridge, 1965
(AASHO)

The study disclosed considerably small difference among these specifications with respect to design load, allowable stress and other factors.
The design was provisionally prepared based on the Specifications for Designing Steel Highway Bridge (Japan Highway Association).

Major conditions taken into account in designing are as given below.

(1) Dead load : TL-20 (equivalent to H-20 in AASHO)

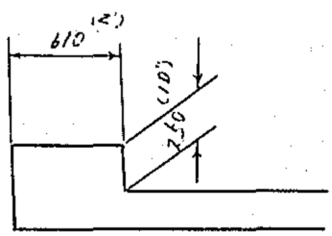
(2) Type
Main span : 3 - span continuous through truss bridge
Side span : 3-span continuous plate girder bridge

(3) Length of Bridge : 713.800 m

(4) Span Arrangement : 4.00+ 3 @ 25.000 + 1.000 + 4 @ 70.000 + 1.000 + 4 @ 70.000 + 1.000 + 3 @ 25.000 + 4.00

(5) Width of Roadway and Sidewalk
Width of roadway : 7.314 (24')
Width of sidewalk : Not included

(6) Curbs

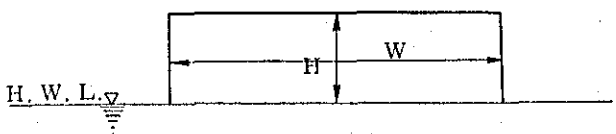


(7) Navigational clearance
W = 45.720 (150')
H = 12.000 (30' + 3.000)

(8) Maximum longitudinal slope
i = 4%

(9) Lateral grade
i = 1.5 %

(10) Seismic coefficient
Kh = 0.05
Kv = 0



(11) Thickness of pavement
t = 50 mm

(12) Temperature range

- 20°C ~ +20°C

(13) Structural steel

All structural steel to conform to JIS specifications.

(14) Minimum thickness of member

t = 8 (5/16")

(B) Outline of Structire

This section presents the outline design of the proposed Gorai River Bridge. Calculations for designing structures were therefore limited to the minimum extent required for estimating the approximate construction cost.

In other words, calculations were confined to those for major cross-sections, and the drawings were prepared with the view to indicating general dimensions and major cross sections.

(1) Items to be taken into account for bridge type selection

Alignment best fit for the smooth flow of traffic and the overall economic advantages derivable from the bridge construction must be taken into consideration in selecting the type of bridge.

The Mission was of the opinion that the execution of the project demands various viewpoints and deep insight.

Items to be taken into account with respect to the superstructure are as follows.

(i) Low cost

Cost for maintenance and repair should naturally be taken into account. It is to be emphasized, however, that the difference in interest accruing from different cost is much larger and more important than the maintenance cost.

(ii) Excellent road utility

Construction should be carried out to achieve the pre-determined alignment. It must be noted, however, that accurate longitudinal alignment cannot sometimes be gurranteed because of the technical difficulty that may be involved in some of bridge structures such as continuous composite girders and P.C. structure which are accompanied by unequal settlement.

Number of expansion joints should be minimized since improper fitting of joints leads to imperfect bridge erection and uneveness in the road surface.

(iii) Easy erection

Bridge construction at a place where the climate is distinctly divided into dry and rainy seasons is often vulnerable to unexpected accidents. Safe and time-saving erection methods should therefore be selected.

(iv) Easy maintenance

With a few exceptions, erected bridges are generally left for maintenance. If, therefore, no periodical and systematic maintenance can be expected, the bridge should be so constructed so as to have a structure that with amply make up for anticipated negligence.

(2) Span arrangement

Cost per 1 m of a bridge is greatly affected by the span arrangement. Therefore, even if the total length of the bridge is given, a large difference in the total construction cost is incurred by the span arrangement. However, span arrangement is not arbitrarily chosen, but is confined by such conditions as navigational clearance and beautility. If the type of one span is decided, it leads to the determination of the girder height and kind of materials to be used for that span, whereby the height and materials of other spans are spontaneously made clear. In such a case, however, the design of the bridge cannot always be the best from economical viewpoint. Span arrangement should therefore be made with careful considerations to the expected economic advantages.

i) Span arrangement for river section

The span arrangement of the proposed Gorai River Bridge was determined with navigational clearance and discharge rate of water taken into account. To be precise, the navigational clearance required for boats sailing under the proposed bridge is, as already described, 45.720 m in width. With the effects of scouring action taken into account, it was disclosed that a minimum span of 70.000 m or more would be required. Each span for the river section was therefore decided to be 70.000 m, and the length of the bridge for the river section was made to be 540.000 m in consideration of the expected erosion of both banks.

ii) Span arrangement for approach part

A span of 25.000 m was determined for approach part considering the future utility value of the clearance under the bridge for navigational purpose. This span opening is considered most economical for a steel bridge of this type.

(3) Clearance below bridge girder

The clearance below bridge girder in main spans were made to be 12.000 m higher than the top of mainmasts of boats sailing under the bridge.

This clearance is considered sufficient since most of boats found in the vicinity of the bridge site are sailingboats.

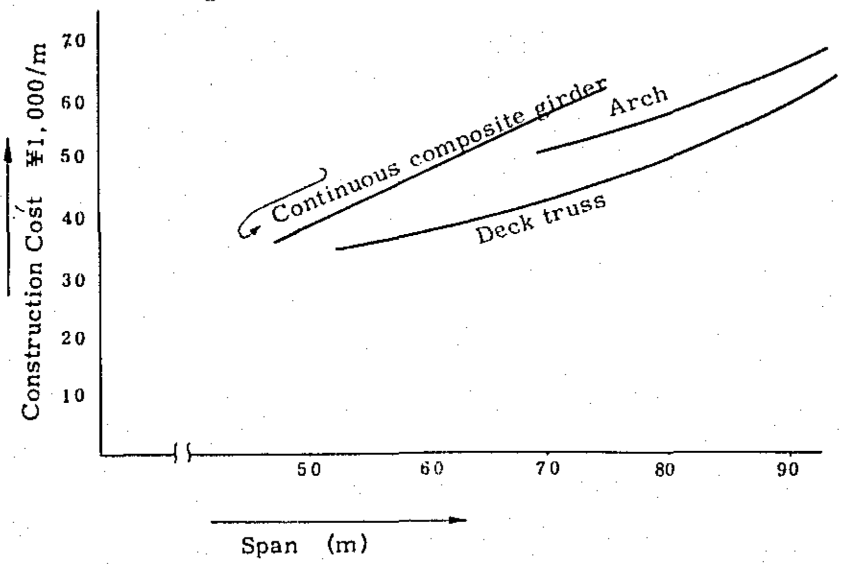
(4) Type of Structure

i) Main section

For the main section, 4-span continuous type was preferred for the sake of road utility and beautility, and truss type was adopted since it ensures easy erection and more economic advantages. A 4-span continuous truss bridge was therefore adopted for crossing the river section.

For the sake of reference, construction cost of bridges with span ranging from 50 m to 90 m is given in the graph below.
The graph clearly indicates that a truss bridge is far more cost-saving if the span is set at 70 m.
The graph below was prepared based on data made available by Japan Highway Public Corporation, and is therefore relative to bridges with larger areas than the proposed Gorai River Bridge. Note that the graph is given merely for the sake of comparison.

Fig. 11 Construction Cost of Superstructure



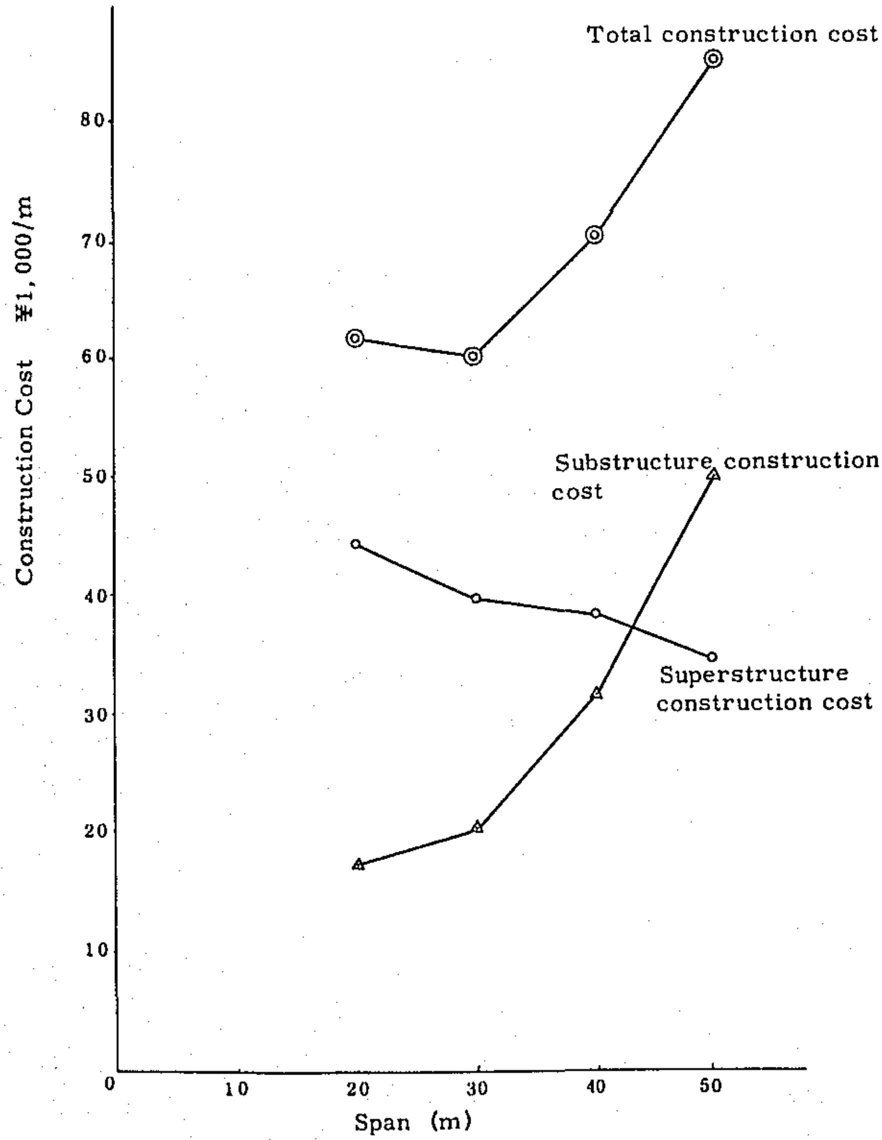
ii) Approach part

In case of a steel bridge, it is generally accepted that shorter span is commendable since it creates less steel weight and therefore saves cost. However, this generally accepted concept does not apply to a place where the foundation conditions are poor and incurs a high cost for construction of the substructure.

Study of alternative designs reveals that a span of about 25.000 m is most economical if piles are driven in the foundation, provided that the cost include expenses for substructure construction. (This explanation is based on comparative data of a bridge designed by Pacific Consultants, Co., Ltd. and erected by the Ministry of Construction)

For the sake of reference again, results of comparison between alternative designs are given in the graph below.

Fig. 12 Relationship between span and cost



(5) Outline of structure

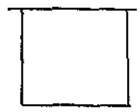
i) Truss bridge

Panel length	: 8.750 m
Main truss interval	: 9.084 m

Max. cross sectional area:

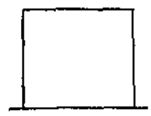
Upper chord member

1-Cov.	Pl.	420 x 14
2-Web	Fl.	360 x 14
1-Bott.	Pl.	350 x 14



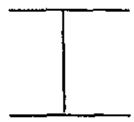
Lower chord member

1-Cov.	Pl.	350 x 14
2-Web	Pl.	350 x 14
1-Bott.	Pl.	420 x 14



Diagonal member

2-Flg.	Pl.	350 x 14
1-Web	Pl.	320 x 10



ii) Plate girder bridge

Maximum sectional area

End-span inner girder

1-Flg.	Pl.	340 x 32
1-Web	Pl.	1,300 x 9
1-Flg.	Pl.	300 x 32

Inner and outer girders on support

1-Flg.	Pl.	320 x 28
1-Web	Pl.	1,300 x 9
1-Flg.	Pl.	380 x 32

Girder height - 1.300 m

Main girder's arrangement - 3.000 m

(6) Steel materials

Steel Material	Allowable Max. tensile stress	Equivalent value in DIN	Equivalent value in AASHO
SS41	1400 kg/cm^2	St. 37	A7 A373
SM50	1900 kg/cm^2	St. 52	A242 A440 A441

3 Substructure

(A) Type of Foundation

As stated in the preceding section, fine and medium sands predominate in the layers at the bridge site with the exception that clayey soil layer having a thickness of about 15.0 m is disclosed only in the boring site B-5. The bearing layer for foundation, i.e., the lower sand layer (refer to the "Estimated Soil Profile") assumes such an aspect as the elevation is about -34m at Faridpur side, around -30.0 m in the neighbourhood of the middle of the river, and about -21 m at Jessore side, ascending from Faridpur to Jessor side on the whole.

As a result of soil exploration and in view of the large load due to superstructure, the foundation for the main span can satisfactorily be made by employing piles or well foundation. This fact is also applicable to the approach span.

With respect to the choice to be made from the designing point of view between the large pore size pipe and the well to be provided for the main span foundation, the well foundation will be marked out if the local materials are to be used to the maximum extent. As for scouring, even if the local scouring takes place in the neighbourhood of the pier, the vertical supporting force will need no perticular attention, but the horizontal supporting force is suspected to be impaired extraordinarily, asking for the necessity of deeper footing.

Although the pile foundation is generally more economical in respect of construction cost than the well foundation, yet in the actualities that the present bridge site is subject to unfavourably large scouring and the steel bridge would be vulnerable to the horizontal external forces, the well foundation is not only economical but safer in construction.

For this reason, the Mission decided to employ the well foundation for the main span which is subjected to large external forces, on condition that bricks be used as aggregate for concrete.

The fundamental type of well foundation to be used for the foundation of the structure is automatically determined depending mainly upon the size and shape of the structure to be loaded on it. But the conventional well foundation mostly uses circular, elliptic, oval, square of rectangular form.

Out of these various forms, the circular type well has the most smallest volume with reference to the bottom area of the foundation and naturally has the smaller friction resistance against earth. Moreover, the circular type well foundation with cutting edges equally arranged in view of its center is beneficial to uniform and fast sinking especially when the excavation is carried out in water. Other advantages in circular type will be clear in the following comparison: as regards stress to be imposed on the well section, the circular type deserves uniform compressive stress but not a bending force; on the other hand, the rectangular type is subject to both the compressive and tensile stresses due to the bending moment, whereby it requires stronger section. Naturally, if made up of identical material, the circular type is stronger and more economical than the square type. The elliptic well is subject to either or both of the compressive and tensile stresses according to its ratio of the major to minor axis; as regards the feasibility of execution, the simpler, i.e., linearer the contour of the framing, the easier and more economical the construction. However, sinking the well requires comparatively fast work and as a result, the concrete will not be allowed for its due curing time. The circular or elliptic type is preferable in this respect. Any structures constructed in the river gives a bad effect to the river flow, and therefore it clearly damages the state of the river. When comparison is made between two factors, i.e., one acting for reducing effective flow area and another for turbulating flow, the circular type is the worst in the section of river and flow, and the square type is most problematic in rectification of the river. Therefore, the ellipse with a comparatively small minor axis is most preferable in this respect. Finally, to make allowance for the bearing force of ground, where the foundation is constructed in the soft ground, the well is intended to strengthen the foundation, and where the ground is rigid, the ground itself becomes the first problem. With a certain fixed length of periphery, the circle has the maximum area. Naturally, if the problem is how to strengthen the bearing force, the circular type is most preferable. With respect to stress resistivity, the circle forms a strong structure even with a comparatively thin shell and makes itself light. As a result, the circular type provides beneficial conditions for feeble ground still more.

However, where the bearing force must be dependent on the friction resistance by reason of feeble ground, the foundation having the maximum peripheral length for a certain fixed area such as the square type is naturally conceived advantageous, yet its effects are rather doubtful because of inevitalbe increase in its dead-weight attributable to stress resistivity.
On the whole, the Mission reached the conclusion that the circular type well is most compatible with the existing circumstances.

On the other hand, as to the foundation of the approach span, its structure will give no problems because of small reaction from superstructure as well as of no scouring action expected; the properly arranged piled foundation is preferred to the well type, not only for economical reasons but also in view of safety structure.

Wooden, reinforced concrete, P.C. steel piles, etc. are now in practial use. Of these, the wooden pile excels others in cost, but it no longer meets generally demanded requirements if the structure on the piles are large.

The recommendable pile is the reinforced concrete pile which is economical and assures faster execution by hammering. The demerits of the reinforced concrete pile are the large dead-weight, large piling resistance, brittleness, unavailability of extension joints, etc.

Nevertheless, the above-cited demerits are not particularly serious problems if the pile length required is not more than 20 m. P.C. and steel piles have many merits in comparison with reinforced concrete pile but are not economical.

For this reason, the economically advantageous reinforced concrete piles have been adopted for pile foundation as far as circumstances permit.

As for the cast-in-place concrete for pile foundation, it will not be employed due to the fact that the reliability to horizontal resistance is doubtful and there is little or no method by which to judge the adequacy of execution.

(B) Design of Well Foundation

(1) Evaluation of N-Value of ground

The well foundation has been determined based on the data obtained by the boring tests. Given in the following table are the factors obtained in various boring sites.

Table 17 Various Factors of Foundation Layer

Boring No.	Location of Boring	Depth from river bed to foundation layer		Composition of Foundation layer	Measured N-Value of foundation layer
		More than + 0 m	Less than ± 0 m		
B - 1	M +170.000	M +7.500	M -35.000	Highly dense medium sand	86
B - 2	M +270.000	M +2.700	M -32.000	Highly dense medium sand	59
B - 3	M +380.000	M +0.940	M -34.000	Dense medium sand	63
B - 4	M +490.000	M -1.300	M -36.000	Highly sense medium sand	58
B - 5	M +650.000	M +7.790	M -23.000	Dense fine sand	61

When it is required to calculate the bearing force, the measured N-value must be corrected. The following is the correction made for location B-4.

i) Correction of N-Value on the basis of the boring rod length

The N-value inclines to show superficial excessive value due to the elastic deformation of rod and nature of soil if the boring rod is long.

For this reason, the following equation is used for correction.

$$N = N' \left(1 - \frac{1}{200} X\right)$$

Where

N' : Measured N = 58

X : Length of rod (m) = 35.62 m

Then,

$$N = 58 \left(1 - \frac{1}{200} \times 35.62 \right) = 48$$

ii) Correction of N-Value against soil nature

The N-value should be corrected in case where there exists very fine sand or silty sand below the ground water having low permeability and the value N becomes large, i.e., if the number of blow is in excess of 15.

$$N = 15 + \frac{1}{2} (N' - 15) \; (N > 15)$$

N' = N-value corrected by the boring rod length = 48

$N = 15 + 1/2 (48 - 15) = 31.5 \doteqdot 32$

(2) Calculation of allowable bearing capacity

The bearing capacity of the foundation layer is calculated by making use of the data obtained at location B-4.

Although a number of formulae may be used, the following is employed for calculation.

("Design of Caisson" contained in specifications for Designing Substructure of Highway Bridge)

$$q_d = \alpha CN_c + \beta T_1 \ BN_r + r_2 D_f N_g$$

Where

C : Cohesion under the well foundation (t/m^2) = 0
T_1: Unit weight of well foundation bottom (t/m^3) = 0
r_2: Unit weight of soil under well foundation (t/m^3) = 1.0

α
β): Shape factor of bottom of well

B : Length of short side of well section (m) = 9.30
D_f: Effective length of well (between ± 0.0 and -10.80)
= 25.20

$$\left.\begin{array}{l} N_c \\ N_r \\ N_g \end{array}\right\} \text{Factor of bearing capacity.}$$

The factors of bearing capacity N_c, N_r and N_g, can be given by the *function of angle of internal friction(ϕ inherent to the nature of soil under the* bottom of well, the angle of internal friction, ϕ), is derived from the N-value.

The estimating equation is given below.

$$\phi = 15 + \sqrt{15N} \qquad N > 5$$
$$= 15 + \sqrt{15 \times 32} = 36.9°$$

The factors of bearing capacity, N_r and N_g, corresponding to ϕ = 36.9°, are contained from the relevant chart as follows.

$$N_r = 42, \qquad N_g = 41$$

For circular section of well foundation, the shape factor are:

$$\alpha = 1.3, \qquad \beta = 0.3$$

Therefore, the ultimate bearing capacity, q_d, becomes as follows provided that the firstterm, αCN_c is neglected because of sandy soil.

$$q_d = \beta T_1 \ BN_r + r_2 D_f N_g$$
$$= 0.3 \times 1.0 \times 9.30 \times 42 + 1.0 \times 25.20 \times 41$$
$$= 117.18 + 1,033.20 = 1.150 \ t/m^2$$

Here, if the safety factor is set at 3, the allowable capacity, q_a, would be as expressed in the equation below.

$$q_a = 1/3\ q_d = 1/3 \times 1.150 = 383\ t/m^2$$

(3) Allowable bearing capacity in case of allowable settlement is taken into account

The structure is expected to sink considerably because the well foundation is selected at sand layer. Generally, the settlement of 25 mm is accepted as may have little or no effect on the superstructure, hence, this value is applied to the present calculation.

The following formulae are employed for the calculation, (refer to "Design Standard of Highways, Japan Highway Public Corporation).

$$S_1 = \frac{S_a}{\left(\frac{2B}{B + 0.3}\right)^2}$$

Where

S_1 : Settlement value of the 30 cm square loaded Board (cm)

S_a : Allowable settlement value of the foundation = 2.5 (cm)

B : Width of short side of the foundation section = 9.3 (m)

$$S_1 = \frac{2.5}{\left(\frac{2 \times 9.30}{9.30 + 0.3}\right)^2} = 0.666\ cm$$

Then, the maximum load strength of bearing foundation is obtained from correlative chart of S_1 and N in bearing foundation.

$$q_s = 29\ t/m^2$$

Though the bearing capacity required for a normal force should be reduced to 1/2 of the maximum load strength, but Terzaghi says that the allowable bearing capacity of pier on the sand layer can be twice the allowable value for the footing on the same layer.

Therefore, for the foundation which must bear the cutting edge of well, the following value is given.

$q_{a'} = qs \times 1/2 \times 2 + D_{f'} r$

Where

$D_{f'}$: Effective length of well (m)

r : Unit weight of the soil above the bearing foundation (t/m^3)

$q_{a'}$: $29.0 \times 1/2 \times 2 + 33.0 \times 1.0 = 62.0\ t/m^2$

As can be seen above, the allowable bearing capacity is determined, satisfying the condition $q_a > q_{a'}$

(4) Study of effective well length

The effective well length is determined by the following formula (refer to "Specification to Designing Substructure of Highway Bridges").

$$\left(\frac{2}{n} C' \ Wb_o\right)\ell^5 - (3\ KW_1)\ell^4 - (9H)\ell^3$$
$$- 12\left(M_o - \frac{4}{n} c' \times Wb_o \alpha ka^3\right)\ell^2 - (24\ \alpha ka^3\ KW_1)\ell$$
$$- (24\alpha\ ka^3\ H) = 0$$

Where

n : Safety factor

c' : Coefficient of passive earth pressure

W_1 : Weight of well per unit length (t/m) = 53.14 t/m

W : Unit weight of soil (t/m^3) = 1.0 t/m^3

$2b_o$: Maximum width of well (m) (Normal to action of earthquake)

2 a : Maximum width of well (perpendicular to action of earthquake)

ℓ : Effective well length, 24.70 m

k : Horizontal seismic coefficient, 0.05

α : Shape factor of bottom well

H : Horizontal force at given ground surface

M : Moment at given ground surface

The following explains about Group 7. (loading condition of AASHO)

n : 1.1

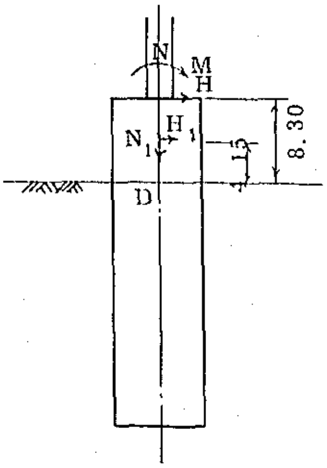
C' : Coefficient of passive earth pressure by Monobe-Okabe's method, 3.1 (from Hand book of Soil Mechanics issued in Japan)

$2b_o$: 10.70 m, 2a : 5.40 m

$$\alpha = 1 - \frac{0.12a}{b_o} = 1 - \frac{0.412 \times 2.70}{5.35}$$

$$= 0.792$$

(5) Various forces to be expected on the given ground surface



$M = 1,993^{t-m}$
$N = 1,727^{t}$
$H = 107^{t}$
$N_1 = 67.21 \times 8.30 = 557.84^{t}$
$H_1 = 557.84 \times 0.05 = 27.89^{t}$

Various forces acting on point 0.

$$\Sigma M_o = 1,993 + 107 \times 8.30 + 27.89 \times 4.15 = 2,996.84^{t-m}$$
$$\Sigma N_o = 1,727 + 557.84 = 2,284.34^{t}$$
$$\Sigma H_o = 107 + 27.89 = 134.89^{t}$$

$$\frac{2}{n} C'Wb_o = \frac{2}{1.1} \times 3.1 \times 1.0 \times 4.65 = 26.2$$

$$3Kw_1 = 3 \times 0.05 \times 67.21 = 10.1$$
$$9\Sigma H_o = 9 \times 134.89 = 1214.0$$
$$12\,(\Sigma M_o - \frac{4}{n} C'Wb_o \alpha ka^3)$$
$$= 12\,(2,996.84 - \frac{4}{1.1} \times 3.1 \times 1.0 \times 4.65 \times 0.589 \times 1.0 \times 4.65^3)$$

$$= -1,288.0$$

$$24\alpha ka^3 Kw_1 = 24 \times 0.589 \times 1.0 \times 4.65^3 \times 0.05 \times 67.21$$
$$= 4,776.28$$

$$24\alpha ka^3 \Sigma H_o = 24 \times 0.589 \times 1.0 \times 4.65^3 \times 134.89$$
$$= 191,719.7$$

$$26.2\ell^{5} - 10.1\ell^{4} - 1,214.0\ell^{3} + 1,288.9\ell^{2} - 4,776.28\ell - 191,719.7 = 0 \ldots\ldots\ldots \text{①}$$

① x 1/26.2

$$\ell^{5} - 0.385\ell^{4} - 46.34\ell^{3} - 49.19\ell^{2} - 182.3\ell - 7,317.55 = 0$$

$\ell = 10.5$

$$127,627.05 - 4,679.7 + 56,943.57 - 1,914.15 - 7,317.55 = 0$$

$\ell = 9.5$

$$77,378.09 - 3,135.85 - 39,73 + 4,439.40 - 1,731.85 - 7,317.55 = 0$$

(6) Allowable stress

The crushed brick as the coarse aggregates will be used for concrete.

<table>
<tr><td colspan="5">Design strength of concrete σ_{28} (kg/cm)</td><td>165</td></tr>
<tr><td rowspan="9">Allowable stress (kg/cm^2)</td><td rowspan="7">Concrete</td><td colspan="3">Compressive stress by bending moment</td><td>55</td></tr>
<tr><td colspan="3">Axial compressive stress</td><td>41</td></tr>
<tr><td rowspan="3">Shearing stress</td><td rowspan="2">In case the diagonal tensil stress is sustained only by concrete</td><td>slab</td><td>5.0</td></tr>
<tr><td>Beam</td><td>3.7</td></tr>
<tr><td colspan="2">In case the diagonal tensile stress is sustained only by reinforcement</td><td>11</td></tr>
<tr><td colspan="3">Bond stress</td><td>4.5</td></tr>
<tr><td colspan="3">Bearing stress</td><td>41</td></tr>
<tr><td rowspan="2">Reinforce-ment</td><td colspan="2">Tensile stress</td><td>SR 24</td><td>1,200</td></tr>
<tr><td colspan="2">Yield point strength</td><td>SR 24</td><td>2,400</td></tr>
</table>

(7) Unit weight of materials

Reinforced concrete	:	2.1 t/m³
Concrete	:	2.0 t/m³
Soil (above ground water level)	:	2.0 t/m³
Soil (below ground water level)	:	1.0 t/m³
Water	:	1.0 t/m³

(8) Dimensions and factors of well

Table 18 Various Factors of Well

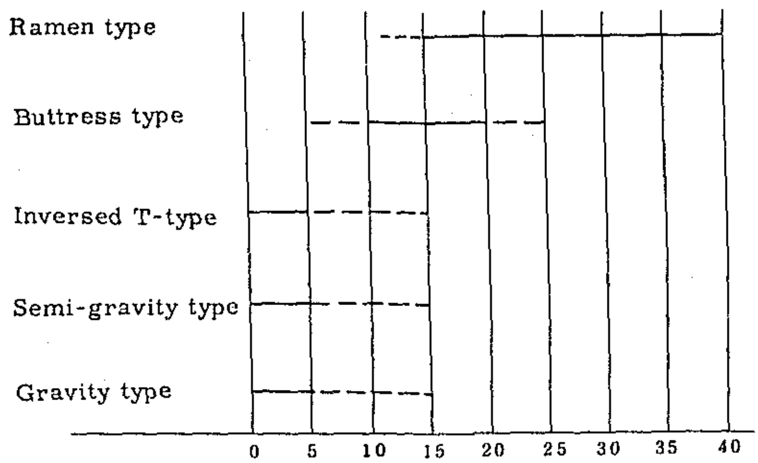
				P. C. Bridge		Steel Bridge	
				M - 4	P - 3	P - 8	P - 3
Diameter			m	9.3	7.4	8.0	8.0
Total length			m	33	32	33	32
Effective length			m	23	22	23	22
Thickness of wall			cm	90	70	80	70
Weight of well			t	2,218	1,316	1,591	1,478
Superstructure Reaction	Dead load		t	1,645	655	1,350	844
	Live load		t	112	104	162	134
	Horizontal force		t	82	34	147	14
	Wind load		t	17	-	74	-
Wave pressure			t	19.4	-	-	-
Bearing Reaction	Normal	Vertical	t/m^2	54.1	47.4	55.3	37.7
		Horizontal	t/m^2	-	-	-	-
	Earth-quake	Vertical	t/m^2	64.2	60.6	65.0	47.0
		Horizontal	t/m^2	-	-	-	-
Allowable bearing force	Normal		t/m^2	62	62	62	62
	Earthquake		t/m^2	82.6	82.6	82.6	82.6
Resistance earth pressure	Normal		t/m^2	1.6	-	-	-
	Earthquake		t/m^2	4.2	1.3	6.4	0.7

(C) Design of Abutment and various Factors

The following types of abutment can be cited for practical use; gravity type, semi-gravity type, inversed T-type, buttress type, box type, ramen type, etc.

Each type of these abutments has a merit to be utilized The economical height of these abutments are illustrated below.

Table 19 Economical Height of Abutment



As is clear in the above graph, the abutment of the present bridge may be recommended to be of the buttress type in view of economy and stability.

Table 20 Various Factors of Abutment

			P.C. Bridge	Steel Bridge
Width		m	8.55	8.55
Height		m	9.40	8.293
Front wall thickness		cm	50	50
Buttress wall thickness		cm	60	60
Footing thickness		cm	100	100
Footing length		m	8	6
Body weight		t	449	318
Super structure reaction	dead load	t	205	70
	Live load	t	37	59
	Horizontal force	t	21	4.0
Earth Pressure		t/m^2	6.86	5.99

(D) Design of Pier and Various Factors

The pier of an oval column type has been selected from the condition that the increase of local errosion might be eradicated and that the determinant pertaining to the selected superstructure is existent.

Also, the outside dimensions of the pier has been standardized both by the forms (P.C. bridge or steel bridge) and by the spans (main and approach spans).

Table 21 Various Factors of Pier

			P.C. Bridge		Steel Bridge		
			P-4	P-2	P-8	P-11	P-12
Vertical width		m	6.4	6.4	6.8(12.0)	6.8 (12.0)	7.1
Horizontal width		m	1.5	1.5	2.2	2.2	1.2
Length of pier		m	19	9.8	22.31	16.79	7.04
Footing dimensions		m x m	-	7.55x 5.00		-	8.6x 3.8
Total weight of bridge		t	246	198	718	557	122
Super-structure reaction	Dead load	t	409	409	632	287	192
	Live load	t	104	104	224	158	106
	Horizontal force	t	21	21	111	37	26

(E) Design of Pile Foundation and Various Factors

Table 22 : Various Factors of Pile Foundation

			P.C. Bridge		Steel Bridge	
			Abut-ment	Pier P - 2	Abut-ment	Pier P -12
Diameter		mm	450	450	450	450
Thickness		mm	70	70	70	70
Total length		m	20	20	20	20
Weight		t/pc	4.34	4.34	4.34	4.34
No. of piles		Pcs	30	24	28	17
Nor-mal	Max. reaction	t/pc	49.2	34.4	38.9	36.2
	Max. horizontal force	"	3.6	-	4.7	-
	Allowable vertical bearing force	"	50	50	50	50
	Allowable hori-zontal bearing force	"	5	5	5	5
At time of earth-quake	Max. reaction	"	55.8	40.2	40.2	50.6
	Max. horizontal force	"	5.5	-	6.0	2.1
	Allowable verti-cal bearing force	"	66.7	66.7	66.7	66.7
	Allowable hori-zontal bearing force	"	6.7	6.7	6.7	6.7

CHAPTER V ERECTION PLAN

CHAPTER V ERECTION PLAN

1. Superstructure of P. C. Bridge

(A) Temporary Facilities

(1) Machinery

(i) Concreting machine

Batcher plant, 0.6 m^3	: 1 unit
Mixer 0.6 m^3	: 1 unit
Concrete bucket 0.6 m^3	: 2 units
Belt conveyor, ℓ=7 m	: 6 units
Flexible vibrator, ϕ 750, 3 PS	: 4 units

(ii) Vorbauwagen

W = 30 t	: 2 units

(iii) Equipment for P. C. use

Jack, Dywidag type (for ϕ 27), 50 t	: 4 units
Bar bender, 5 PS	: 1 unit
Grout mixer, 1,500 rpm, 3PS	: 1 unit
Grout diaphragm pump	: 3 units
Compressor, 3 PS	: 1 unit
Electric welder, 15 kw	: 1 unit
Dynamometer	: 1 unit

(iv) Concrete testing machines

100 t Amsler compression tester	: 1 unit
Mold slump tester	: 1 unit
Aggregate sieving tester	: 1 unit
Grout flow precipitation tester	: 1 unit

(v) Simple cable crane - 1 set

Power : 30 PS, double-driven winch : 1 unit

Main rope : ϕ 25 wire

Running rigging : ϕ 16 wire

Span : 150 m

Main post : Tower H = 25 m H = 50 m

Max. load : 4.0 t

(vi) Machines for girder erection

Erection girder

(Length 42 m, Extension length 25 m, Height 1.5 m, Capacity 30 t) : 2 sets

Hydraulic jack, for girder, 40 t	: 8 units
Journal jack, 25 t	: 4 units
Lever block, 5 t	: 10 units
Heavy trolley, 50 t	: 4 units
Wire rope, ϕ 18	: 1,000 m
Rail, sleeper	: 300 m
Winch, 20 PS, double-drum type	: 2 units
Grooved rail for traversing girder (incl. steel ball)	: 20 m

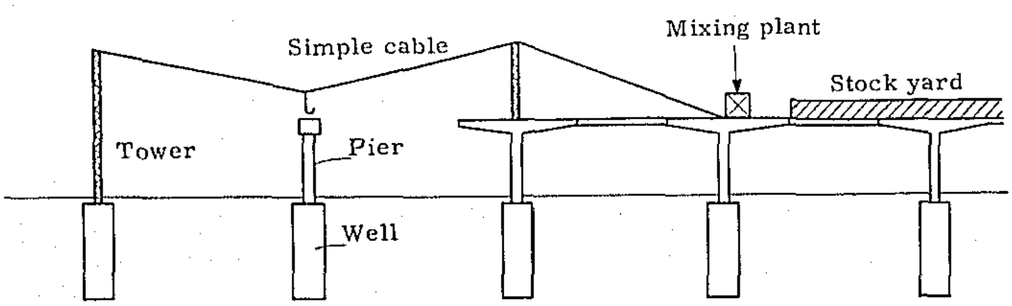
(vii) Electric power facility

(viii) Water facility

Pump	: 2 units
Hose	: 100 m

(2) Erection method

Fig. 13 Erection Method of P.C. Bridge



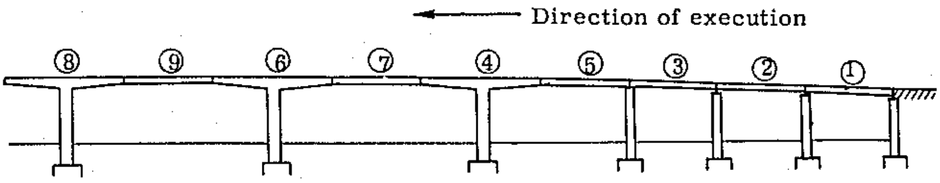
In order to facilitate shifting the temporary erection materials and stockyard as the execution progresses, the girder beam or finished road surface is utilized.

The simple cable is used not only for transporting concrete and other materials but also for assembling the Vorbauwagen.

(B) Erection Order of Superstructure

Since the simple cable is used for handling material and assembling of Vorbauwagen as explained above, the erection is to be carried out from one side.

Fig. 14 Erection Method of P.C. Bridge



In another method such as Gerber method, some portion of bridge is completely established first and then the suspension girders are set. This is quite difficult because of possibility of flood and the high tapper adopted.

(C) Installation of Overhang Girder

The mobile scaffolding called "Vorbauwagen" is mounted on the beam to carry out forming, reinforcing, concrete placing and prestressing works. The Vorbauwagen is composed of two major parts: one having the timbering function to support the form and the placed concrete, the rhombic frame part; and another having the scaffolding for work.

Loads are supported by the jack installed near the centre of the rhombic frame, and the tensile force caused in the frame back is cancelled changing into the prestressing force which is supplied to the finished girder through the P.C. rod . In addition, Vorbauwagen can travel on the rail.

Erection Order

(i) Assemble the timbering on the pier, and place concrete on top of the pier by means of the simple cable.

(ii) Assemble the Vorbauwagen (hereinafter referred to as "wagen") on top of the pier, and carry out the concrete placing up to the No. 2 block.

(iii) By this time, No. 1 wagen has been shifted with the work up to No. 2 block being completed. Therefore, on top of the pier, the space to assemble No. 2 wagen can be obtained.

After No. 2 wagen is assembled, carry out the two-block execution on the opposite side of No. 1 wagen.

(iv) Thereafter, the work is alternately performed while the extending arms are balanced.

(D) Erection Method of Suspension Girder

When the overhang parts are constructed, the suspension girders are to be mounted from one side of the river on the premises of the erection method.

In this method, howerver, the pier is subjected to the bending moment unless the counterweight is hung at the tip of the finished overhang girder.

Fig. 15 Erection Order

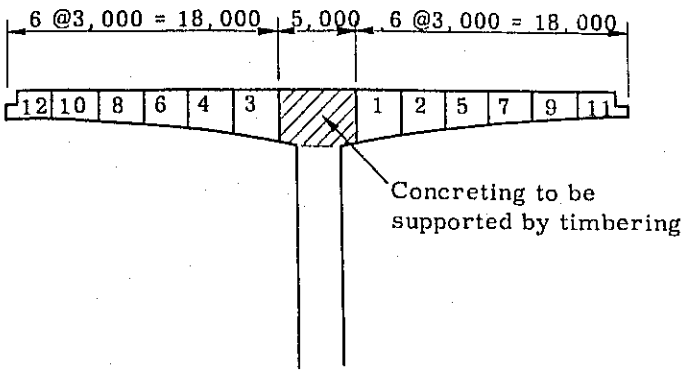
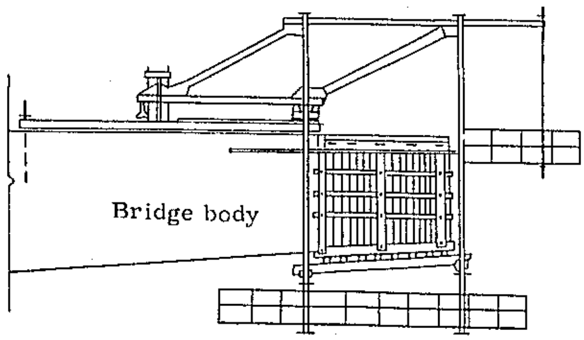


Fig. 16 Vorbauwagen



The two types of counterweight can be used; one is the box girder containing water and the other is the concrete blocks. The former is preferable to the latter in that it is not only economical but also excellent in workability as against the latter which requires a considerably heavy weight (50 t). On the other hand, the erection methods of suspension girders are as instructed below.

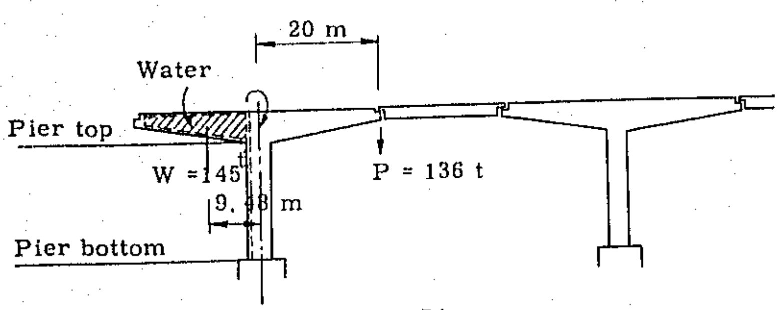
(i) Mount three pieces of suspension beams over the span between the intended and adjacent piers first, fill the beam joint with concrete and place the web beam, and apply the lateral prestress. After finishing one part, the beams for next span are assembled.

(ii) For the purpose of minimizing the bending moment to be created in the piers, two pieces of suspension beams are placed on each span first, then assembling of another beam is carried out for each span by using premounted beams.

The above two methods are taken into account. However, the latter case unavoidably requires the counterweight, too, and has demerits of difficulty in working as explained in the foregoing section 2.

For this reason, the erection of the proposed bridge is carried out in the order of: three pieces of beams are mounted on the first span and after applying the lateral prestress, the work is shifted to the next span. In this procedure, water is used for the counterweight.

(1) Bending moment and axial force expected in case one side of the bridge pier is loaded (excl. felloe guard, handrail, pavement, and cast-in-place concrete for overhang)



Pier top - Bending moment, M = 136 x 20 - 145 x 9.48 = 2,720 - 1,375

= 1,345 tm

Axial force, N = 136 + 145 + 594 = 875 t

Pier bottom - Bending moment, M = 1,345 tm

Axial force, N = 136 + 145 + 1,145 = 1,426 t

(2) Axial force and bending moment of normal loading condition

Pier top - Bending moment, M = 2,285 tm

Axial force, N = 1,206 t

Pier bottom - Bending moment, M = 2,285 tm

Axial force, N = 1,757 t

As can be seen above, the pier is strong enough to sustain forces incurred at time of erection.

(3) When adding one more pier of suspension beam under the state of item (1) above, the load P' of one piece of the suspension beam is set at 50 t by making allowance for impact load and other loads expected.

N' = 50 t, M' = 50 x 20 = 1,000 tm

Resultant: (1) + (3)

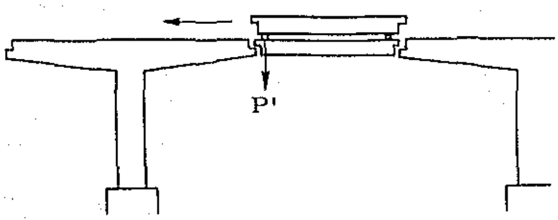
Pier top - M = 1,345 + 1,000 = 2,345 tm

N = 875 + 50 = 925 t

Pier bottom - M = 2,345 tm

N = 1,426 + 50 = 1,476 t

In this case, since the load P' acts for a very short period, the allowable stress of concrete and reinforcement may be considered to be increased 30% more than in normal loading. The above values show enough strength as compared with the normal loading value.

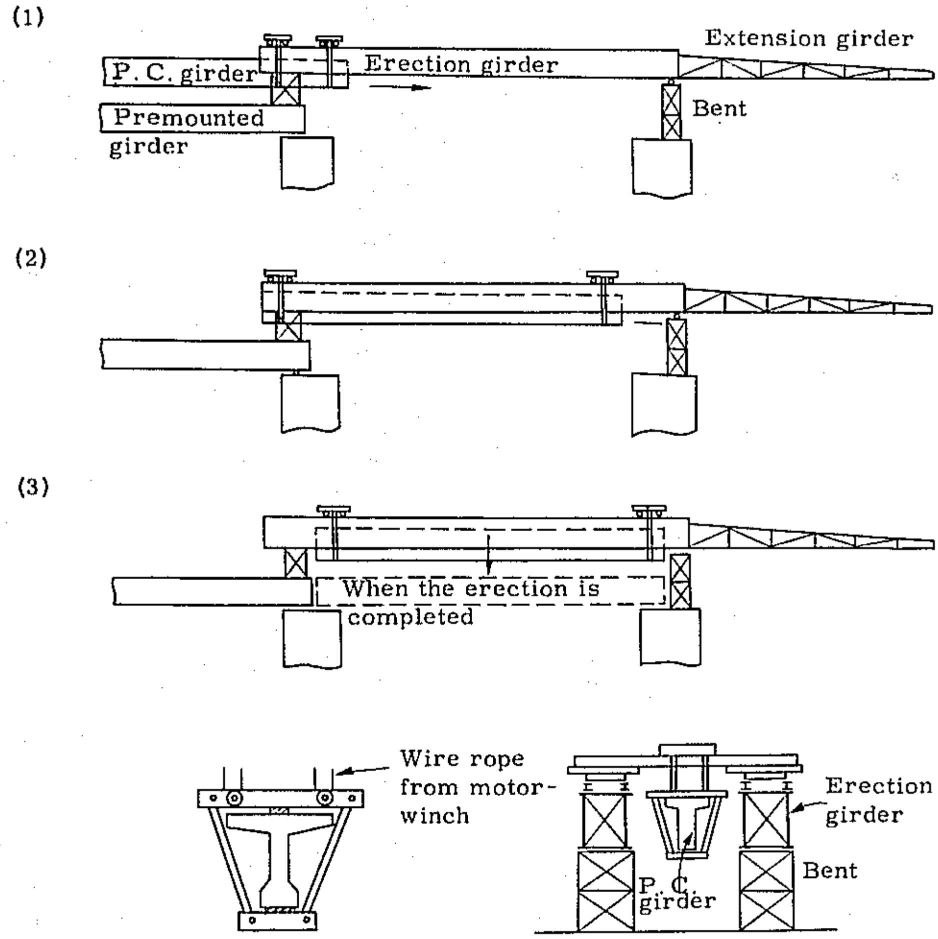


(E) Erection by Erection Girder

In erecting simple beams, the following methods may be used: sliding erection, embrace erection by cable, and erection by erection girder.

With the embrace erection, the erection technique and skill of a reasonable level are necessitated. The erection of the present bridge is therefore to be undertaken safely by the erection girder. On the other hand, the construction of the approach part may be either by staging drawing method.

Fig. 17 Erection Method of P. C. Beam

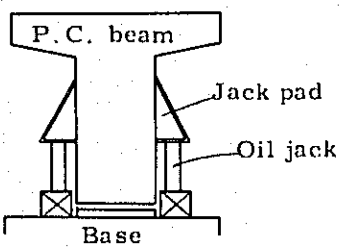


(1) Manufacture of girder

The P. C. beam is manufactured in the first place on the beam making bases (2 bases) which are temporarily installed on the approach bridge. In the course of time, the bases are to be shifted to the near place of the erection.

(2) Adjustment in up and down of beam

When the girder is to be mounted on the transfer car from the beam making base or when the beam is installed in place, the jack pads are fitted on both sides of the beam, and the jacks (oil jack - 4 sets, 4 t) are mounted in order to lift or lower the girder.

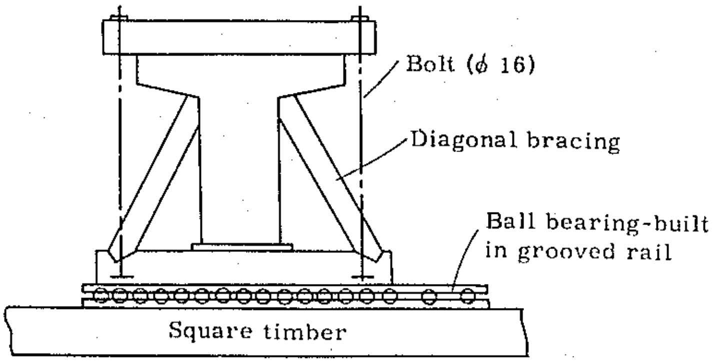


Weight of centre beam : 69 t

Weight of outside beam: 82 t

(3) Lateral movement

For lateral movement of the beam, the diagonal bracing shown in the following figure is fitted to the beam and the beam is placed on to the ball bearing-built in grooved rail.



(4) Erection by erection girder (embrace type)

In this method, the erection is so carried out that the P. C. beam is supported between the two beams as shown in the figure.

The front part of the P. C. beam projecting in form of the bridge is lifted by the front hoist from the transfer trolley; the rear part of the beam is forwarded by the motor-trolley as it is remained on the transfer trolley; when it comes to the position where the rear hoist is stationed, the motor-trolley is once stopped; after the rear part of the beam is lifted, the motor-trolley proceeds to move and stops when it reaches a specified position.

Then, the beam is unloaded on the mount for setting with caution to shock. After the diagonal bracing for protecting turnover is attached to the beam, the beam is loaded on the ball bearing-built-in grooved rail to be shifted in the lateral direction.

2. Superstructure of Steel Bridge

(A) Continuous Plate Girder

The continuous girder is to be installed on the ground. Therefore, the bent is used for the erection work . Out of the girders which are arranged in the stock-yard, the larger ones are transported to the erection site by the trailer while the smaller ones by the truck, and the girder is lifted at its both ends by the crawler cranes (2 units, lifting capacity 10. 2 t per unit), and then unloaded onto the steel bent temporarily, then assembled.

(B) Continuous Truss Girder

As in the case of the above plate girder, this girder is transported to the erection site by the trailer or the truck from the stockyard. Its land side is erected by making use of the bent and its water side by the cable.

(1) Land side (between P3 and P4, and between P10 and P11)

(i) Install the guy derrick for cargo handling at the apporach on the P3 (P11 side).

(ii) Stretch the carrier cable between P3 and P4 (between P10 and P11).

(iii) Install the bent under the truss panel joint.

(iv) Take in the bridge material from P3 (P11), transfer it towards P4(P10), then erect the truss step by step.

(2) Water side

(i) Lay the trolley track on the girder in the land side. Carry in the bridge material.

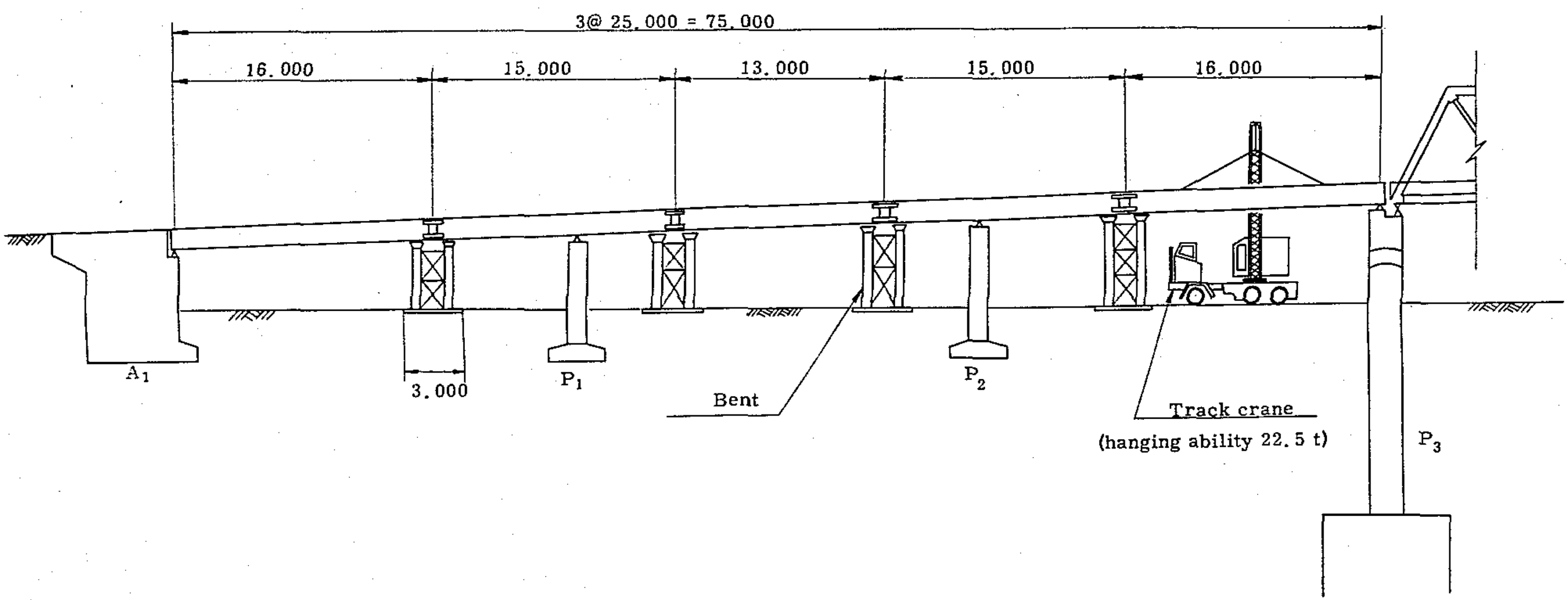
(ii) Stretch the main cable and carrier cable between P4 and P5 (between P10 and P9).

(iii) Erect the truss by nearing from both siedes of P4 and P5 (P10 and P9) towards the centre.

(iv) The spans, P5 - P6, P6 - P7, P7 - P8, and P8 - P9, are erected in the same way as above.

This method is refered to Fig. 18.

Fig. 18 Erection Method of Approch Bridge



(C) Erection Machine

Table 23 Erection Machine for Truss Bridge

Name	Particulars	Q'ty	Remarks
Guy derrick	Boom 250 m; Mast, 30.0 m	1 unit	Use for cargo handling
Travelling crane	Boom, 150 m; Hoisting capacity, 5 t	1 unit	
Bent	7 m (average height), (400 x 300)	As per extension	Use for each erection
Trolley rail	15 kg/m rail	As per extension	Use after changing of preparatory plan
Gate tower	25 m (700 x 700)	2 units	2 units are installed, as a pair, at one span, and use after changing of preparatory plan
Carrier cable	JIS No. 6, 50 ϕ	As per extension	
Main cable	JIS No. 6, 50 ϕ	As per extension	Use after changing of preparatory plan
Carrier cables	10.0 ton	2 units	"
Wires		1 set	"
Blocks		1 set	"
Trolley car	50 ton	1 unit	"
Sleepar	Normal, 140 x 200 x 400		Use after changing of preparatory plan
Winch	30 HP (W), 15 HP (S)	3 units each	Use after changing of preparatory plan
Compressor	20 HP	2 units	"
Power supply		80 kw	Simultaneous operation of 30 HP winches
Tools		1 set	

3. Substructure

(A) Coffering of Foundation

In coffering the main foundation (9 places), the following method is used: steel sheet piles are driven along the circular guide; H-piles are driven in circular as internal waling members; packings and jacks are installed between the sheet piles and the ring beam; and the ring beam is pre-loaded.

Coffering of foundation for approach is carried out as follows: I-beam steel piles are driven in to conform to the shape of foundation; wooden web plates are lined.

(B) Well Sinking Work

Sinking the well is dependent almost on excavation. Howerver, because the well adopted was light weight in comparison with the conventional ones, either one of the methods, counterweight sinking or water jet method, may be adopted.

The excavation of the inside of the well on the land side is carried out by hand until the ground-water appears.

The excavation below ground-water level is by the clamshell only, necessary machines and materials for excavation being handled by cargo ships. The sinking rate per day is estimated at about 80 cm. The forms to be used are made of wood.

(C) Piling

Piling is carried out by Diesel pile driver.

The estimated piling efficiency is 7 pieces (length 20 m) per day.

(D) Construction of Pier and Abutment

The dry processes employed in placing concrete for piers and abutments. Wooden forms are used and scaffoldings are of the steel make. In concreting, ships or concrete tower are to be used.

(E) Temporary Work

Coffering:		
Pilling boat	D - 22	4 unit
Crane boat	30 t	3 unit
Tag boat	90 PS	3 unit
Tripod derrick crane	30 HP	3 unit
Winch	15 HP	4 unit
Well work:		
Clamshell	0.6 m^3	5 unit
Compressor	100 HP	4 unit
Tripod derrick crane	30 HP	3 unit
Concreting:		
Batcher plant		3 sets

Dump truck	7.5 t	6 unit
Concrete tower and winch	50 HP	6 unit
Compressor	75 HP	2 unit
Concrete vibrator		20 unit
Drinage work:		
Drain pump	ϕ 180 mm	10 unit
Cleaning work:		
Earth transporting barge	100 m^3	2 unit
Dump truck	7.5 t	10 unit

Piling:

Rotary dermack	2 unit

Temporary offices:

Construction office	: 124 m^2
Supervisor's office	: 99 m^2
Watchbox, 2 places (i.e., one each on either side of the river)	: 27 m^2
Supervisor's accomodations	: 480 m^2
Machinery storehouse (2 houses, one each on either side of the river)	: 168 m^2
Cement storehouse (2 houses, one each on either side of the river)	: 178 m^2
Carage (for trucks, jeeps)	: 80 m^2
Oil storehouse	: 13 m^2
Power receiving station	: 10 m^2
Labourers' house	: 770 m^2
Wood-working shop	: 110 m^2
Iron-working shop	: 60 m^2

CHAPTER VI ESTIMATION OF CONSTRUCTIONAL MATERIALS

CHAPTER VI ESTIMATION OF CONSTRUCTIONAL MATERIALS

1. Superstructure of P.C. Bridge

(*Overhang girder inclusive of pier)

Table 24 Constructional Materials (1)

Material			Unit	Overhang girder (7 sets)	Simple girder (14 sets)	Total
Con-crete	σ28 350	Concrete	m^3	1,668.8	1,310.4	2,979.2
		Cemene	t	617.5	484.8	1,102.3
		Sand	m^3	767.6	602.8	1,370.4
		Gravel	m^3	1,535.3	1,205.6	2,740.9
	300	Concrete	m^3		204.4	204.4
		Cement	t		65.4	65.4
		Sand	m^3		92.0	92.0
		Gravel	m^3		184.0	184.0
	240	Concrete	m^3	2,623.5	407.4	3,030.9
		Cement	t	734.6	114.1	848.7
		Sand	m^3	1,180.5	183.4	1,363.9
		Gravel	m^3	2,361.0	366.7	2,327.7
Form			m^2	17,833.8	9,630.6	27,514.4
P.C. cable		Longitudinal tightening, ϕ45	kg		63,651.6	63,651.6
		Lateral tightening, ϕ35	kg		11,597.6	11,597.6
P.C. bar, ϕ27			kg	105,130.2		105,130.2
Fitting			Set	3,934		3,934
Coupler			Pcs	5,754		5,754
Sheath		Longitudinal tightening, ϕ45	m	24,853.5	16,459.8	41,313.3
		Lateral tightening, ϕ35	m		5,346.6	5,346.6
Cone		Longitudinal tightening, ϕ7	Pcs		1,260	1,260
		Lateral tightening, ϕ5	Pcs		1,876	1,876
Reinforcement SR24			kg	139,020	96,046	235,066
Asphalt pavement			m^3	177.8	266.0	443.8
Handrail			t			16.8
Extension device			t			
Shoe			t			19.15
Drain device (vynil pipe)			kg			627.0
Timbering			m^3	7,474.7		7,474.7

2. Substructure of P.C. Bridge

Table 25 Constructional Materials (2)

Material				Unit	Quantity
Body (Abutment, 2; Pier, 13)	Concrete	σ 28 165 (Reinforced)	Concrete	m^3	1,137.0
			Cement	t	410
			Sand	m^3	380
			Brick	m^3	350
		σ 28 165 (non-reinforced)	Concrete	m^3	30.5
			Cement	t	11
			Sand	m^3	10
			Brick	m^3	10
	Reinforcement SR24			t	78.7
	Brick for basement			m^3	48.2
	Form			m^2	2,947.94
	Timbering			m^3	4,219.96
	Excavation volume			m^3	1,526.06
	Backfill volume			m^3	1,068.24
Well Foundation	Concrete	σ 28 165 (Reinforced)	Concrete	m^3	5,844.45
			Cement	t	2,110
			Sand	t	1,930
			Brick	t	1,800
		σ 28 165 (Non-reinforced)	Concrete	m^3	1,384.05
			Cement	t	500
			Sand	t	460
			Brick	t	430
	Reinforcement			t	414.5
	Steel material			t	1.8
	Form			m^2	14,527.4
	Timbering			m^3	1,493.6
	Excavation volume			m^3	19,750.8
Pile	Centrifugally cast reinforced concrete pile, ϕ450 x 20.00			Pcs	150

3. Superstructure of Steel Bridge

Table 26 Constructional Materials (3)

Material	Unit	4-span continuous truss, 2 sets	3-span continuous girder, 2 sets	Total
Total steel weight	t	860,752	135,138	995,890
Concrete	m^3	1,077	291	1,368
Cement	t	359	97	456
Sand	t	485	131	616
Gravel	t.	959	259	1,218
Reinforcement SR24	t	179	48	227
Form	m^2	2,304	622	2,926
Asphalt pavement	m^3	206	56	262
Balustrade	t			64.2
Extension device	t			11.5
Shoe	t			50.1
Drain device (vinyl pipe)	kg			627

4. Substructure of Steel Bridge

Table 27 Constructional Materials (4)

Material				Unit	Quantity
Body (Abutment 2, Pier, 13)	Concrete	σ28 165 (Reinforced)	Concrete	m^3	4,361.5
			Cement	t	1,570
			Sand	m^3	1,440
			Brick	m^3	1,350
		σ28 165 (Non-reinforced)	Concrete	m^3	24.1
			Cement	t	9
			Sand	m^3	8
			Brick	m^3	8
	Reinforcement			t	217.1
	Brick for basement			m^3	48.2
	Form			m^2	6,627.1
	Timbering			m^3	12,442.7
	Excavation volume			m^3	1,341.3
	Backfill volume			m^3	425.4
Well Foundation	Concrete	σ28 165 (Reinforced)	Concrete	m^3	4,759.1
			Cement	t	1,720
			Sand	m^3	1,570
			Brick	m^3	1,470
		σ28 165 (Non-reinforced)	Concrete	m^3	1,256.6
			Cement	t	460
			Sand	m^3	420
			Brick	m^3	390
	Reinforcement			t	368.9
	Steel material			kg	1,629.0
	Form			m^2	12,553.2
	Timbering			m^3	1,384.6
	Excavation volume			m^3	14,351.6
Pile	Centrifugally cast reinforced concrete pile, ϕ450 x 20.00			Pcs	124

CHAPTER VII ESTIMATED CONSTRUCTION COST AND CONSTRUCTION SCHEDULE

CHAPTER VII ESTIMATED CONSTRUCTION COST AND CONSTRUCTION SCHEDULE

1. Conditions for Estimating Construction Cost

Conditions to be taken into account in estimating the construction cost are as described below.

(i) Construction of substructure is to be executed, due to its structure, chiefly by the foreign contractor who should give technical cooperation and consultations to local contractors. The substructure is to be erected mainly by the local contractors with consultations provided by the consultant as occasion demands.

(ii) For the purpose of cost estimation, a Japanese firm is assumedly to be assigned as the contractor.

(iii) Estimated construction cost is to be indicated in US dollars irrespective of its being payable in local currency or foreign currencies. Items for which cost is to be estimated are as given below.

Material cost (in foreign currency) - to cover steel materials, PC cables, forms, electrical equipment.
(in local currency) - to cover materials for concrete, fuel.

Labour cost (in foreign currency) - to cover expenses required for the stay of technicians of the foreign contractor in East Pakistan.
(in local currency) - to cover payment to local contractors.

Equipment cost (in foreign currency) - to cover depreciation of constrution machinery and equipment.
(in local currency) -

Freight and transportation cost (in foreign currency) -to cover travelling expenses and freight between Yokohama and Chittagong.
(in local currency) - to cover expenses for transportation between Chittagong and bridge site.

Taxes (in foreign currency) -
(in local currency) - Import tax (10 % of the value of imported equipment and materials)
Income tax (3% of the income of the foreign contractor)
Corporation tax (1.2% of the total cost precluding above taxes and miscellaneous expenses)

Miscellaneous expenditure (in foreign currency) - About 10% of the total cost precluding taxes.
(in local currency) -

(iv) 8 months are estimated to be required for preparation of construction drawing and for tenders, and 31 months for transportation of necessary equipment to the bridge site for construction. A total of 39 months are therefore assumed to be required for the overall construction period.

(v) Power supply required for the construction is assumed to be made available in Faridpur.

(vii) Details of Estimation Items

1. Freight charge

(1) Chittagong - Bridge Site (payable in local currency)

US$2 per ton for one way transportation of construction machinery and materials.

(2) Yokohama - Chittagong (payable in foreign currency)

US$45 per ton for one way transportation of construction machinery;

US$25 per ton for one way transportation of construction materials.

2. Miscellaneous expenses

(1) Payable in foreign currency

Approximately 10% of the total construction cost excluding taxes and consulting fees.

3. Fee for local contractors

(1) Payable in local currency

Approximately 10% of expenses payable in local currency, excluding taxes.

4. Consulting fee

(1) Payable in foreign currency

Expenses for preparation of detailed drawings; approximately 3.5% of the total cost excluding taxes.

(2) Payable in foreign currency

Expenses for supervision of construction; $120 per day per engineer.

5. Taxes

(1) Payable in local currency

Import tax: 10% of the value of imported machinery.

(2) Payable in local currency

Income tax: 3% of labour cost of workers belonging to the foreign contractor.

(vi) Unit cost of major materials and labour cost

Table 28 Unit Cost of Material and Labours

Item		Unit cost	Remarks
Material Cost	Construction steel (t)	470 US$	
	Reinforcing bars (t)	120	
	Steel sheet piles (sheets)	300	
	R.C. piles (Pcs)	150	
	Wooden forms (m^2)	6	
	Steel forms (m^2)	4.4	
	Steel timbering (m^2)	2	
	Cement (t)	36	
	Sand (m^3)	4	
	Gravel (m^3)	14	
	Crushed brick (m^3)	5	
	Brick (m^3)	4	
	Asphalt (m^3)	7	
	P.C. steel cables (t)	360	
	Bearing shoe (t)	415	
Labour Cost	Steel (t)	22	
	Piling (RC piles) (Pcs)	3.5	
	Moulding work (m^2)	0.2	
	Timbering work (m^3)	0.1	
	Steel sheet pile work (sheets)	5	
	Concreting work (m^3)	3	
	Excavation work (m^3)	1	
	Sinking work (m)	20	
	Prestressing work (t)	132	

(3) Payable in local currency

Corporation tax: 1.2% of the total construction cost excluding taxes and miscellaneous cost.

Table 29 Construction Cost of P.C. Bridge

Note: F.C. = Foreign currency
L.C. = Local currency

(US$)

		Substructure		Superstructure		Temporary facilities		Total		
		F. C.	L. C.	F. C.	L. C.	F. C.	L. C.	F. C.	L. C.	Total
Local contractors fee			42,760		59,400		9,270		111,430	111,430
Consultants fee		12,540		170,350				282,890		282,890
Cost for Facilities	Construction machinery & equipment	80,000		148,000		10,000		238,000		238,000
	Buildings						84,720		84,720	84,720
	Power supply					2,000	3,000	2,000	3,000	5,000
	Water supply						5,000		5,000	5,000
	Others									
Cost of Materials	P.C. steel material			85,350				85,350		85,350
	Reinforcement		59,160		64,650				123,810	123,810
	Steel sheet pile					120,000		120,000		120,000
	R.C. pile		22,500						22,500	22,500
	Form		104,850		165,050				269,900	269,900
	Timbering	11,430		14,950				26,380		26,380
	Cement		109,120		73,330				182,450	182,450
	Sand		11,120		11,190				22,310	22,310
	Gravel				78,300				78,300	78,300
	Crushed brick		12,950						12,950	12,950
	Brick		190						190	190
	Asphalt			500	1,300			500	1,300	1,800
	Steel materials			48,880	8,850			48,880	8,850	57,730
Labour cost			81,130		158,730				239,860	239,860
Power charge	Electricity charge									
	Fuel charge		5,500		4,000				9,500	9,500
Freight charge		84,250	21,070	60,500	28,600			144,750	49,670	194,420
Miscellaneous expenses		64,600			101,160	23,400		88,000	101,160	189,160
Sub-total		352,820	470,350	528,530	754,560	155,400	101,990			
		823,170		1,283,090		257,390				2,363,650
Taxes	Import tax		8,000		14,800		1,000			23,800
	Income tax		3,380		5,110					8,490
	Corporation tax		9,100		15,290		3,090			27,480
TOTAL		357,820	490,830	528,530	789,760	155,400	106,080			
		843,650		1,318,290		261,480				2,423,420

Table 30 Construction Cost of Steel Truss Bridge

Note: F.C. = Foreign currency
L.C. = Local currency

(US$)

		Substructure		Superstructure		Temporary facilities		Total		
		F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	F.C.	L.C.	TOTAL
Fee for local contractor			46,410		21,730		9,270		77,410	77,410
Fee for consultants		115,000		154,000				269,000		269,000
Cost of Facilities	Construction machinery & equipment	80,000		157,400		10,000		247,400		247,400
	Buildings						84,720		84,720	84,720
	Electric power					2,000	3,000	2,000	3,000	5,000
	Water supply						5,000		5,000	5,000
	Others									
Material Cost	Steel materials			528,300	14,100			528,300	14,100	542,400
	Reinforcement		70,320		40,900				111,220	111,220
	Steel sheet pile					120,000		120,000		120,000
	R.C. pile		18,600						18,600	18,600
	Form		115,080	5,900	6,400			5,900	121,480	127,380
	Timbering	27,660						27,600		27,600
	Cement		135,330		22,000				157,330	157,330
	Sand		13,750		1,300				15,050	15,050
	Gravel				8,300				8,300	8,300
	Crushed brick		16,090						16,090	16,090
	Brick		190						190	190
	Asphalt			500	1,300			500	1,300	1,800
Labour cost			67,090		88,100				155,190	155,190
Power charge	Electricity charge									
	Fuel charge		6,000						6,000	6,000
Freight charge		91,000	21,690	63,500	34,900			154,500	56,590	211,090
Miscellaneous expenses		70,920		97,290		22,470		190,680		190,680
Sub-total		384,580	510,550	1,006,890	239,030	154,470	101,990			
		895,130		1,245,920		256,460				2,397,510
Taxes	Import tax		8,000		15,740		1,000			24,740
	Income tax		3,450		4,600					8,050
	Corporation tax		9,890		13,520		2,810			26,220
TOTAL		384,580	531,890	1,006,890	272,890	154,470	105,800			
		916,470		1,279,780		260,270				2,456,520

2. Time Schedule of Construction

Table 31 Time Schedule of Construction for P.C. Bridge

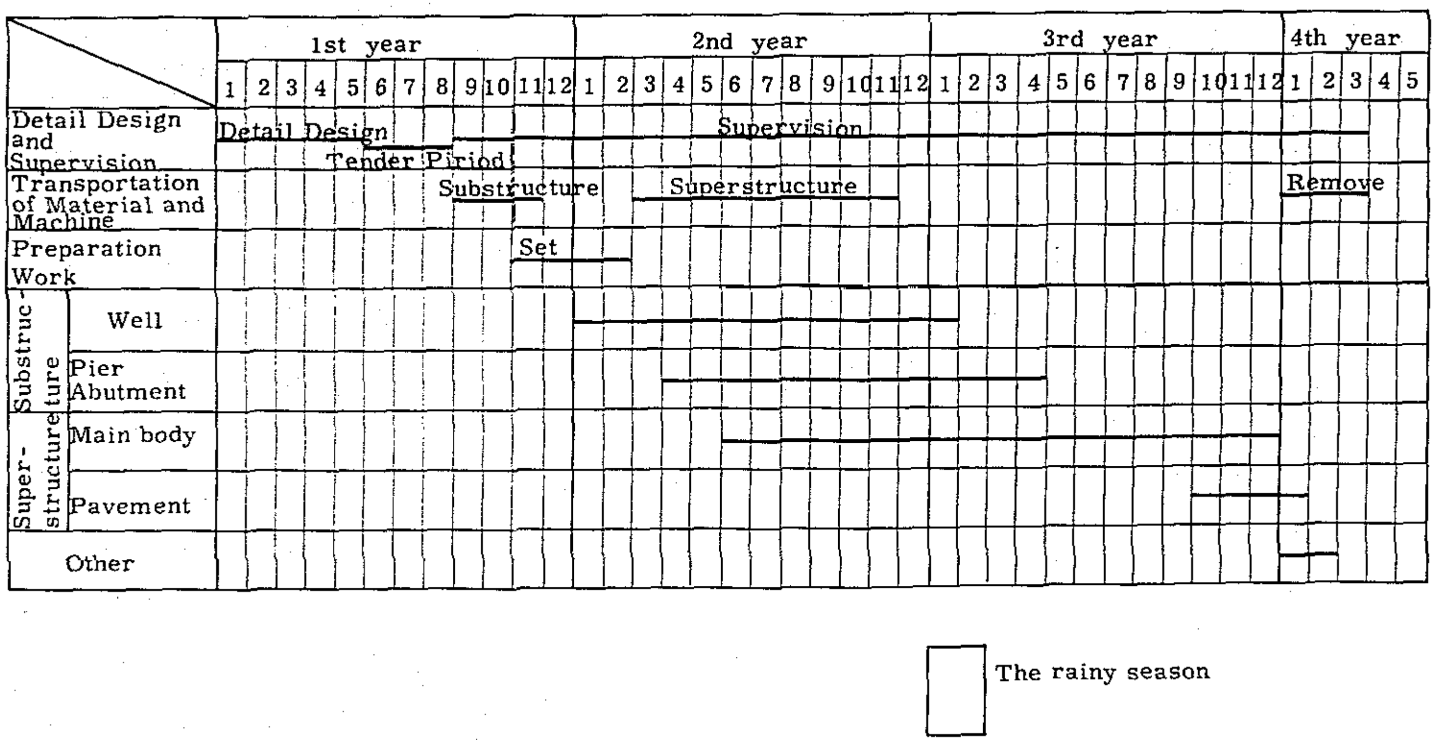
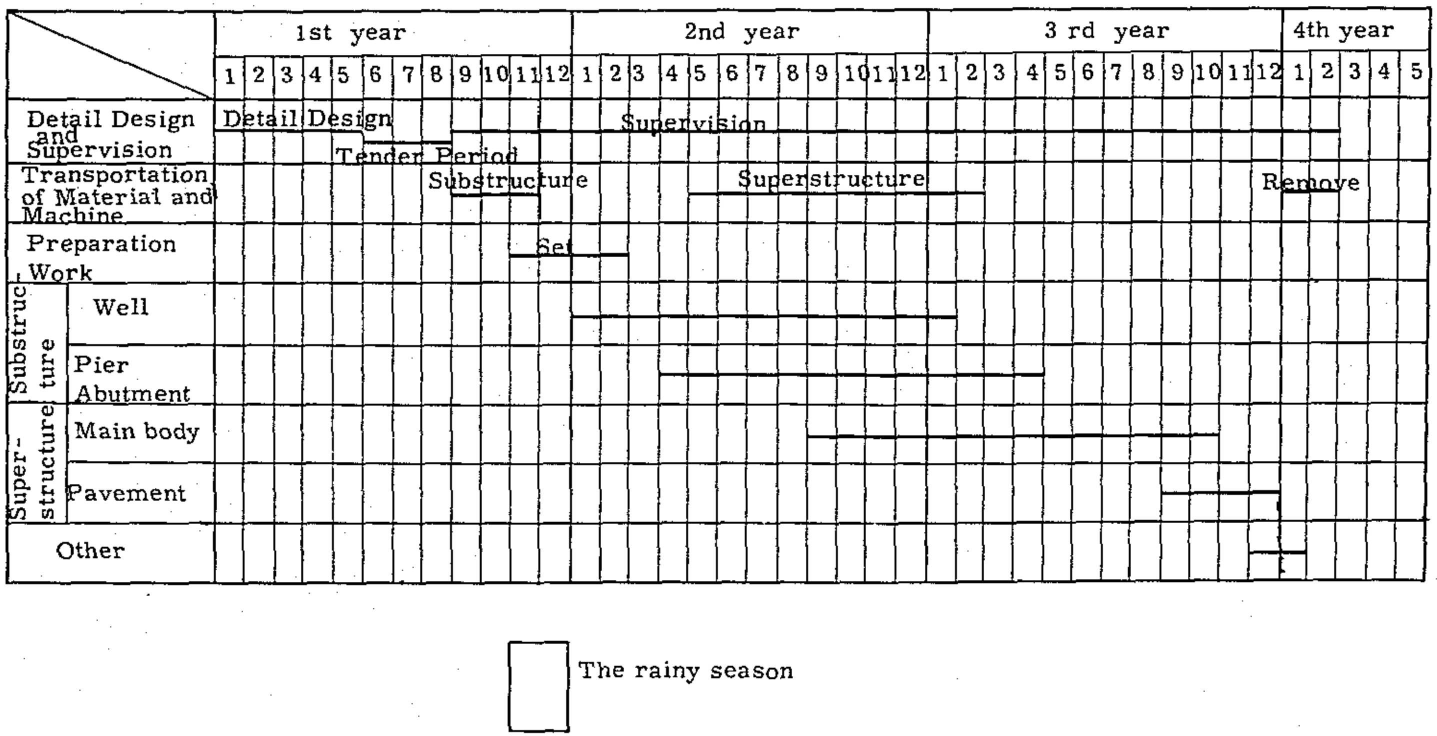


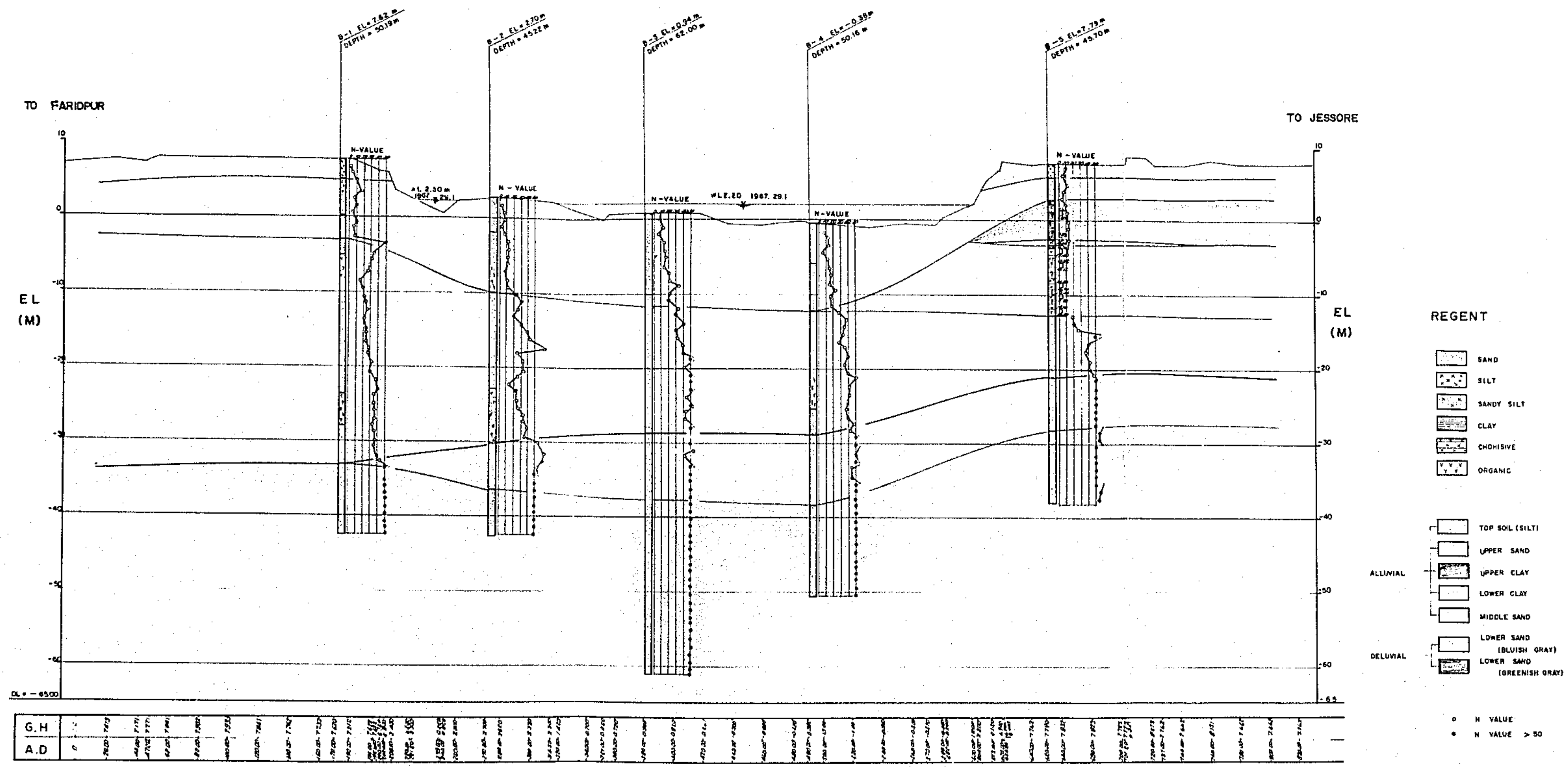
Table 32 Time Schedule of Construction for Steel Truss Bridge



SOIL PROFIL OF PROPOSED GORAI RIVER BRIDGE SITE B

UPPER STREAM SITE

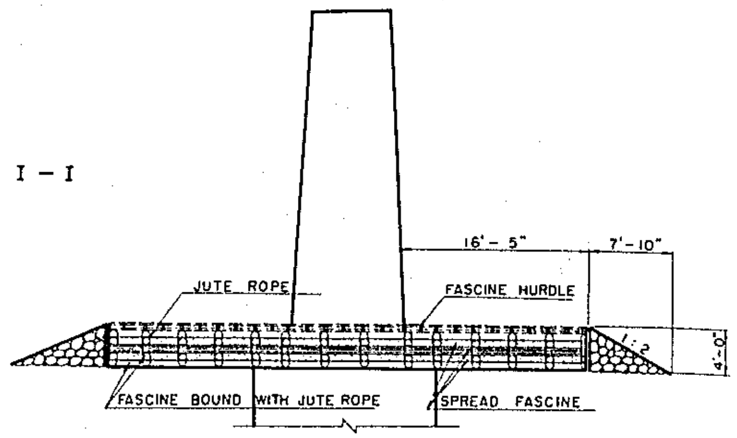
SCALE V = 1: 400, H = 1: 2,000



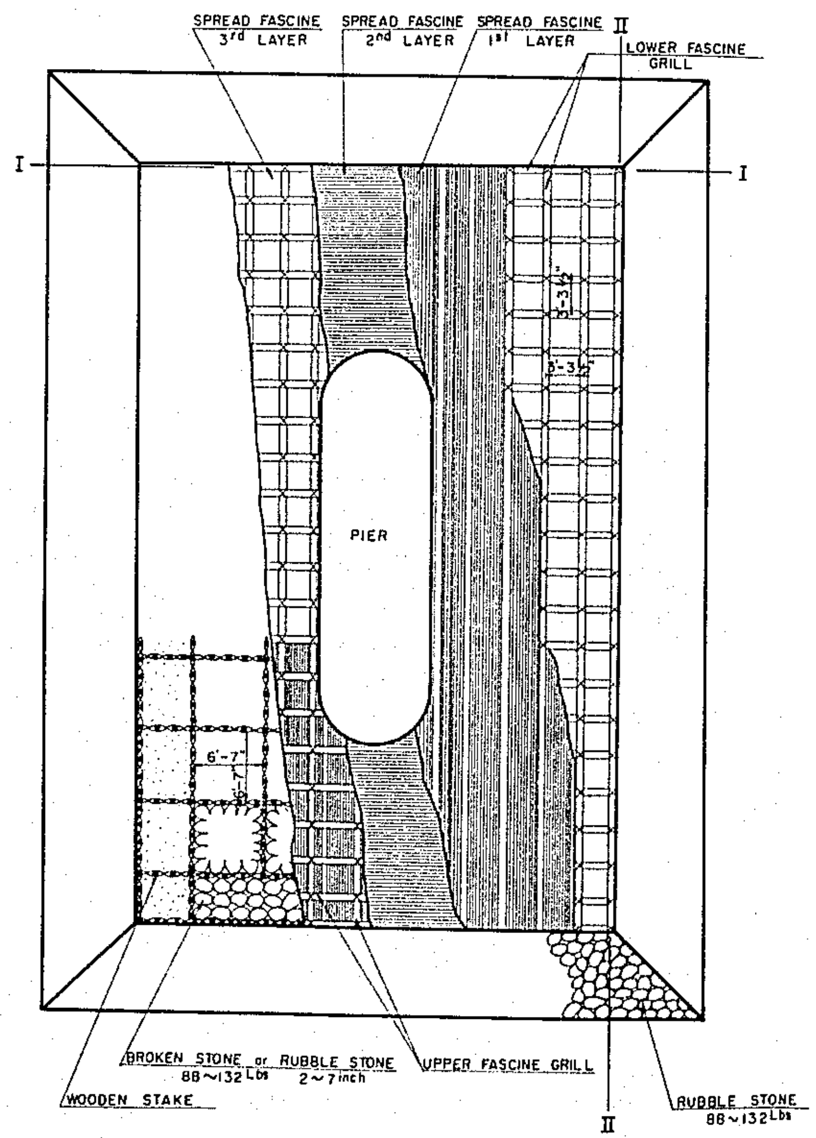
FASCINE MATTRESS IN SUBSTRUCTURE OF SHIN-TORIGAI BRIDGE

SCALE 1 : 100

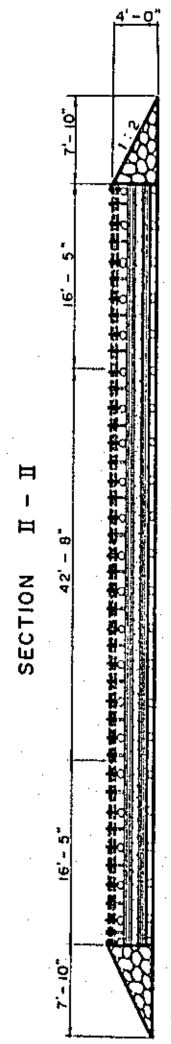
SECTION I - I



PLAN



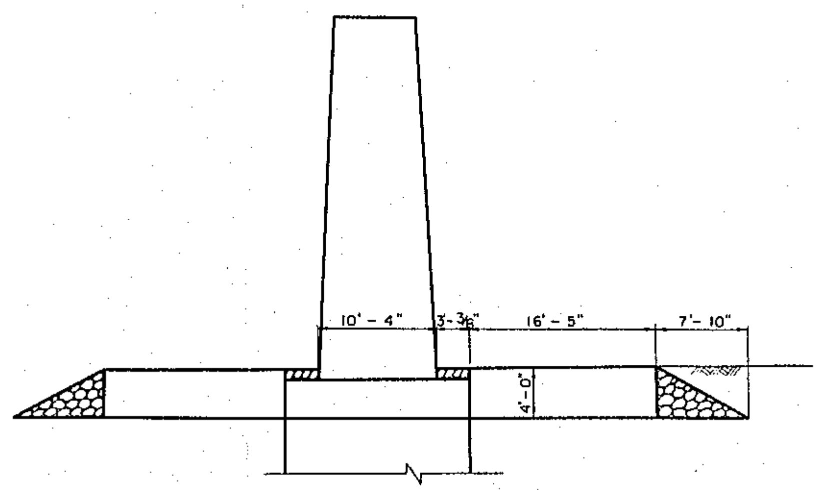
SECTION II - II



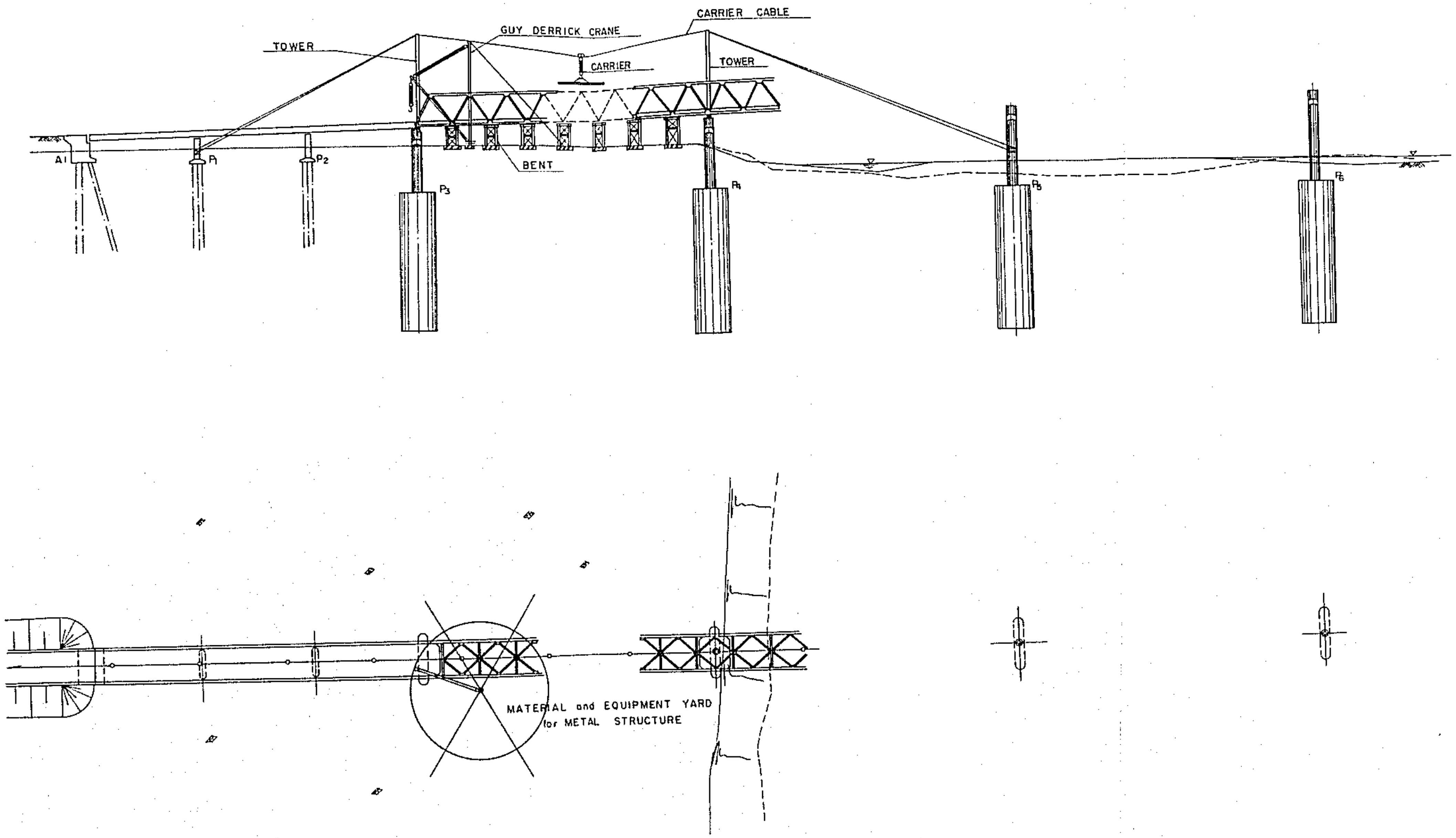
MATERIAL LIST FOR FASCINE MATTRESS FOR $10 yd^2$

ITEM	SIZE	unit	amount for mattress 4' th.	amount for mattress 2'-7" th
FASCINE BUNDLE	40FT Long or more Circumference at 1'-6" from trunk end should be 2'-2"	bundle	60	42
FASCINE HURDLE	ℓ= 52'-6" a bundle consist of 25 twigs	"	2.0	2.0
WOODEN STAKE	ℓ= 4'-0" a bundle consist of 10-pieces	"	2.4	2.4
VINE PIECES or Wire	ℓ= 100' dia. 1/2" (or 1/16" dia of steel wire)	ℓbs.	11	2.2
JUTE ROPE	dia 1/16"	"	11	6
"	dia 1/8"	"	3.3	1.0
RUBBLE STONE	projection length 1'-2"	ea.	135	——

SECTION OF PIER



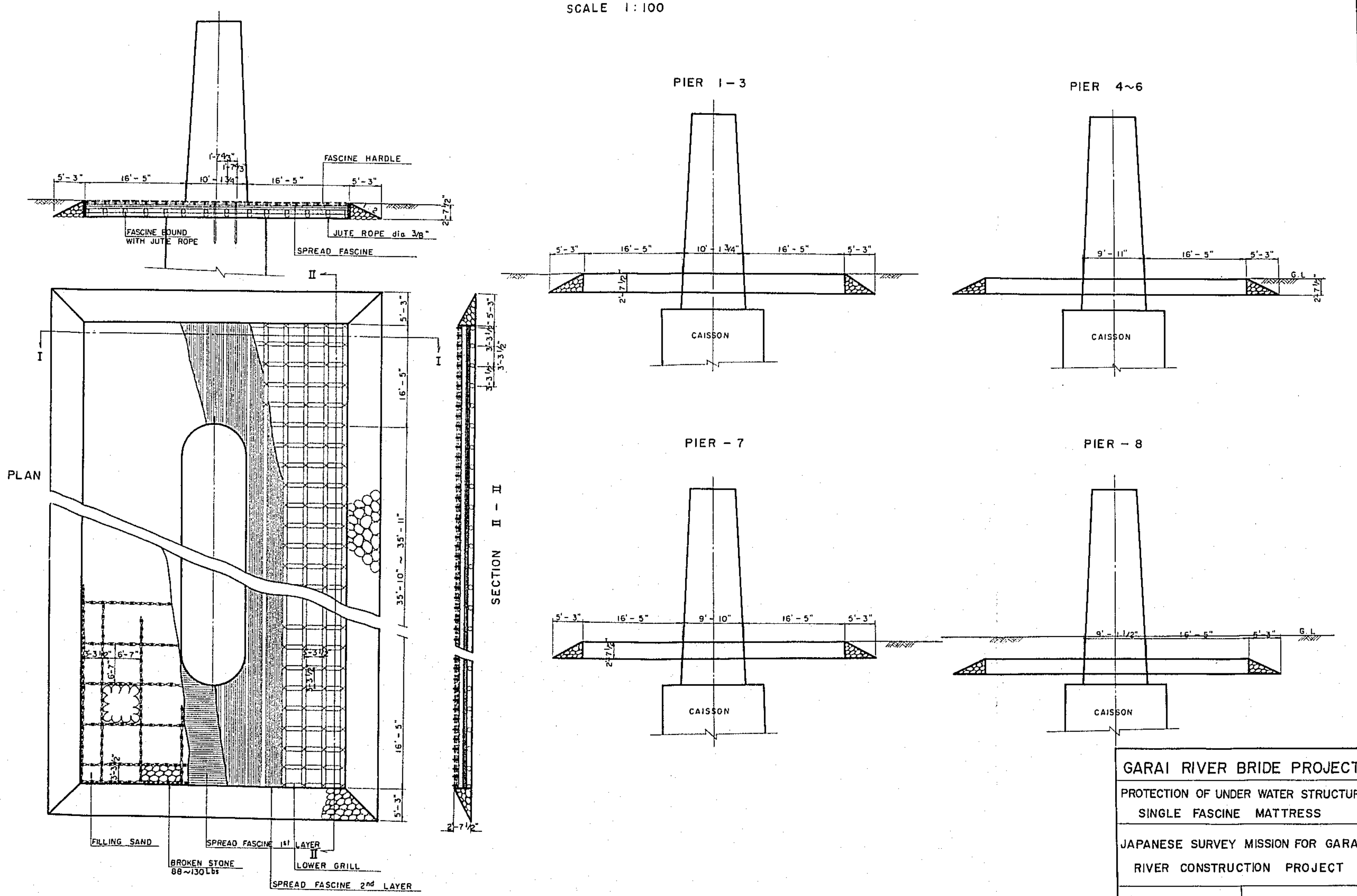
GARAI RIVER BRIDE PROJECT	
PROTECTION OF UNDER WATER STRUCTURE FASCINE MATTRESS	
JAPANESE SURVEY MISSION FOR GARAI RIVER CONSTRUCTION PROJECT	
JUNE 1966	



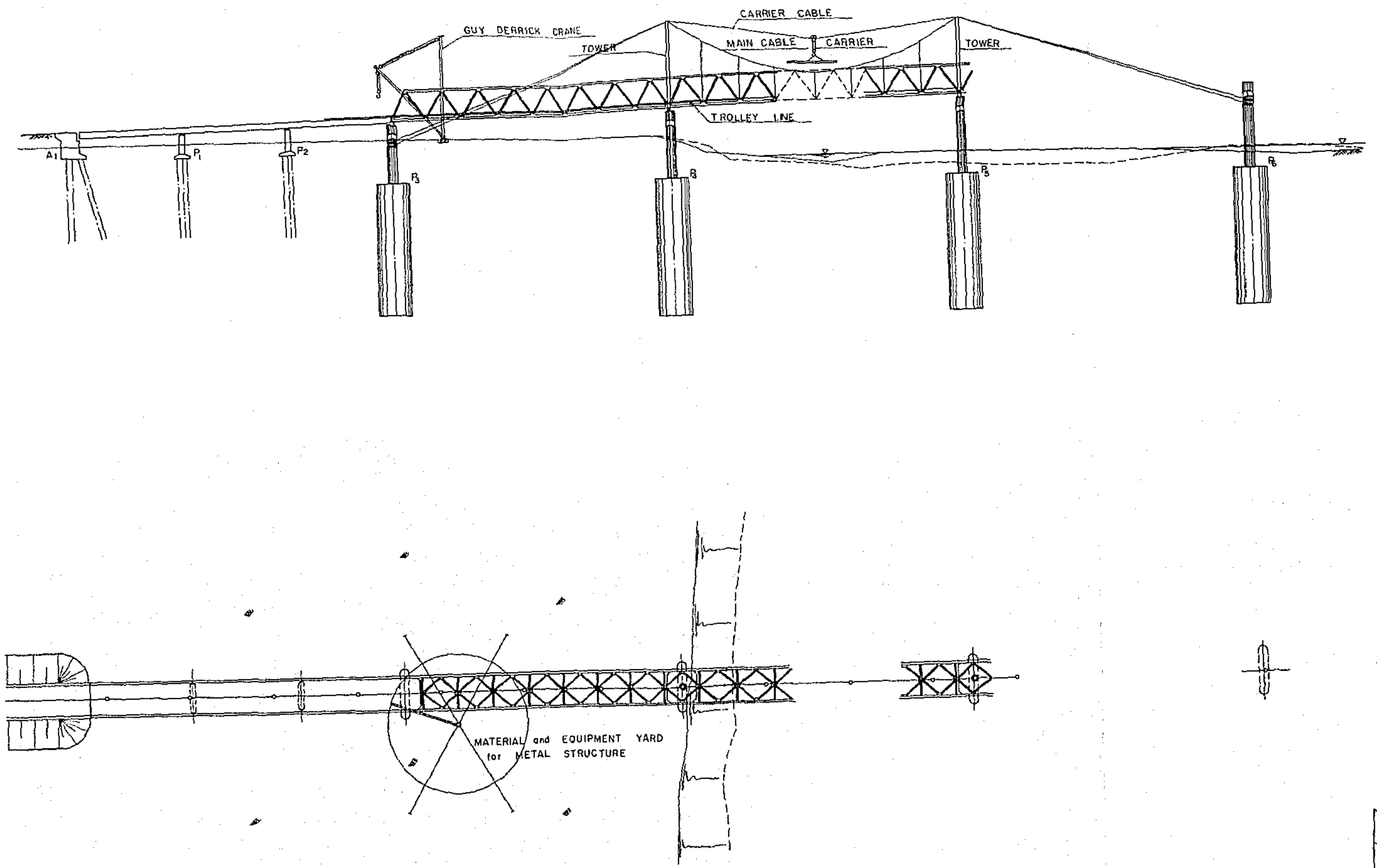
EAST PAKISTAN	
GARAI RIVER BRIDGE PROJECT	
TYPICAL DETAIL STEEL BRIDGE ELECTION (I)	
CHIEF ENG.	SCALE. 1 : 500
CHECKED BY.	SHEET NO.
DESIGNED BY.	
DRAWN BY.	DATE.

SINGLE FASCINE MATTRESS IN SUBSTRUCTURE OF SHIN-TORIGAI BRIDGE

SCALE 1 : 100



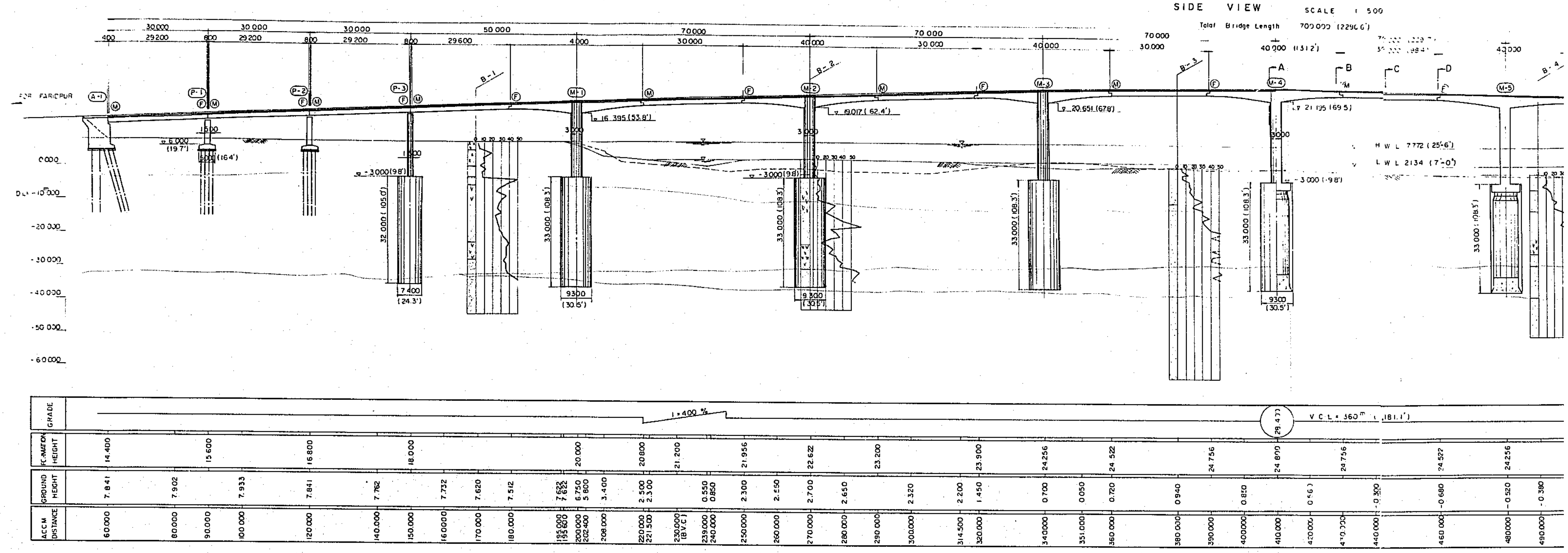
GARAI RIVER BRIDE PROJECT	
PROTECTION OF UNDER WATER STRUCTURE SINGLE FASCINE MATTRESS	
JAPANESE SURVEY MISSION FOR GARAI RIVER CONSTRUCTION PROJECT	
JUNE 1966	



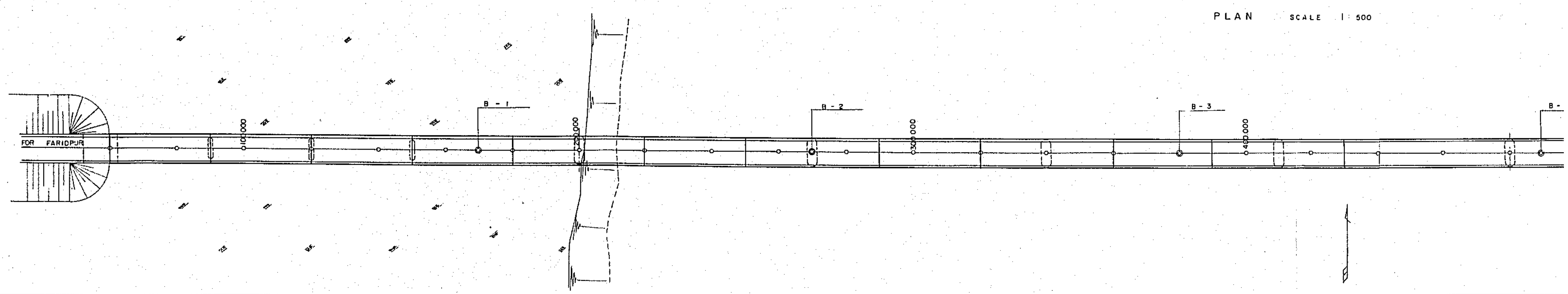
EAST PAKISTAN	
GARAI RIVER BRIDGE PROJECT	
TYPICAL DETAIL STEEL BRIDGE	
ELECTION (2)	
CHIEF ENG	SCALE 1 : 500
CHECKED BY	
DESIGNED BY	
DRAWN BY	DATE

GENERAL PLAN OF GARAI RIVER BRIDGE

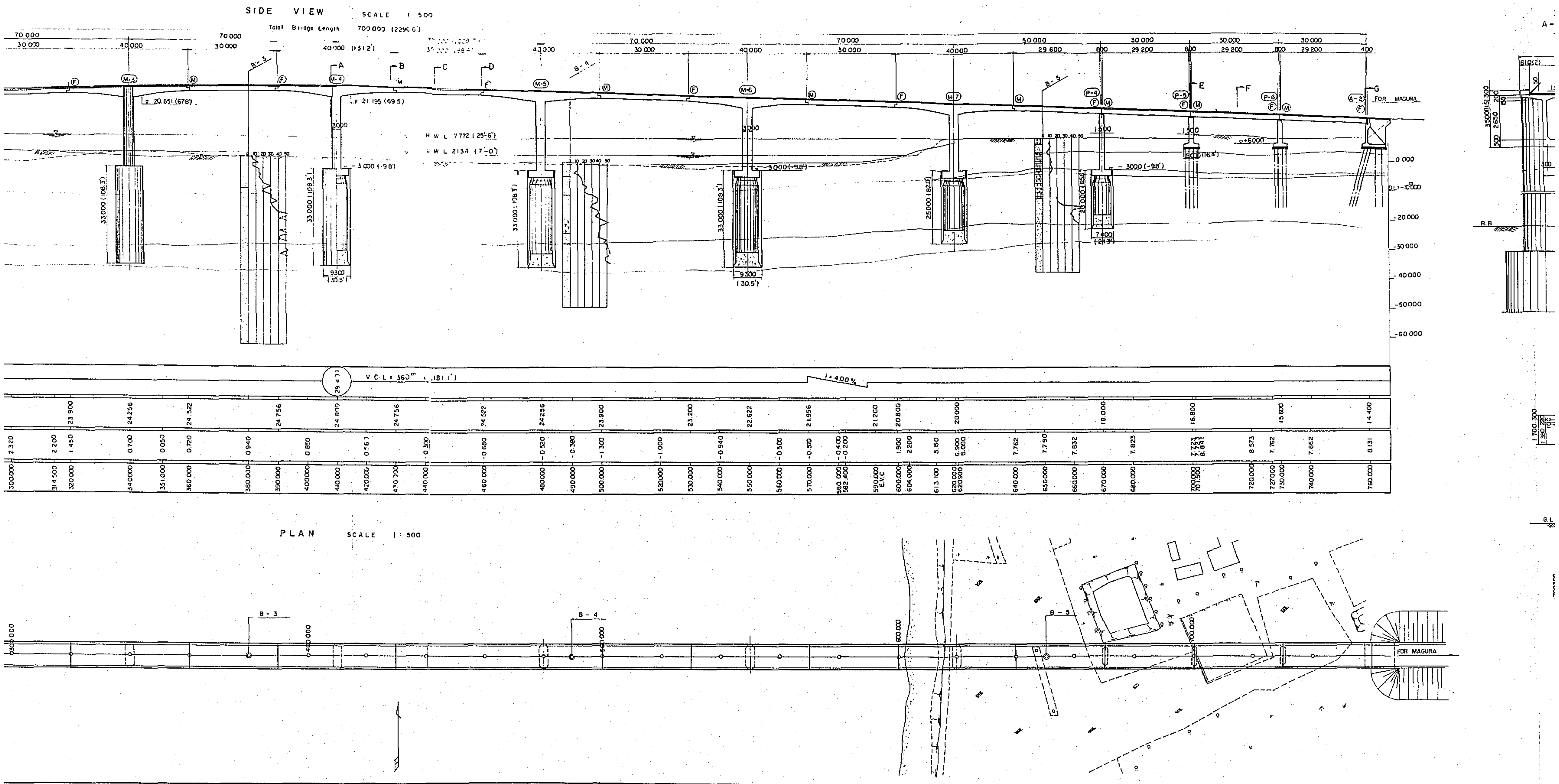
SIDE VIEW SCALE 1:500

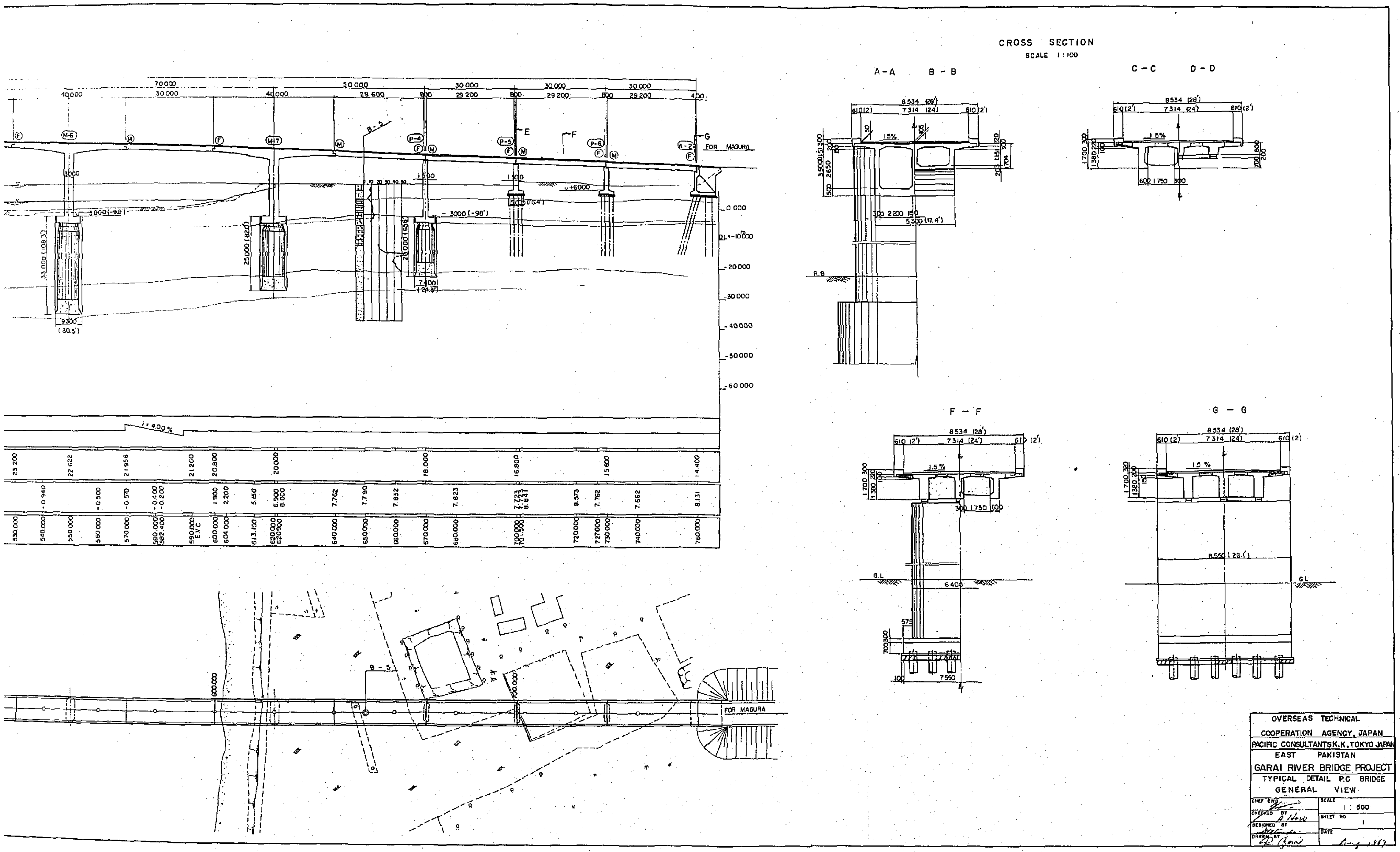


PLAN SCALE 1:500



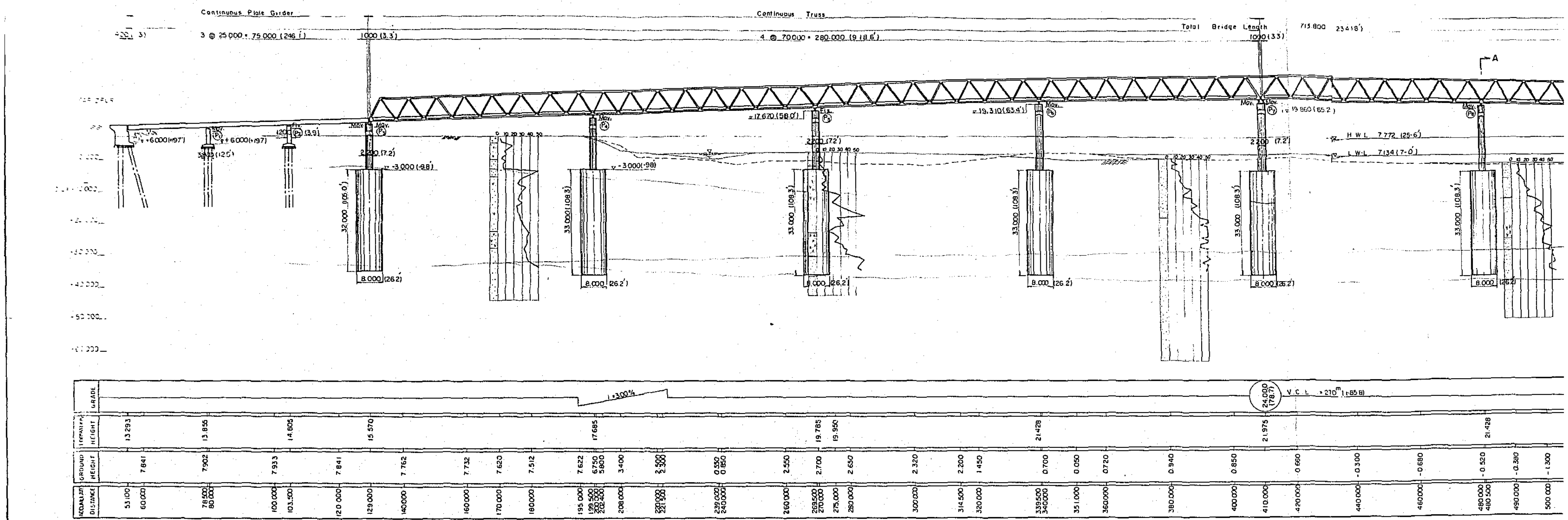
GENERAL PLAN OF GARAI RIVER BRIDGE




CROSS SECTION
SCALE 1:100
A-A
B-B
C-C
D-D
F-F
G-G
FOR MAGURA
OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
PACIFIC CONSULTANTS K.K. TOKYO JAPAN
EAST PAKISTAN
GARAI RIVER BRIDGE PROJECT
TYPICAL DETAIL P.C BRIDGE
GENERAL VIEW
CHIEF ENG.
CHECKED BY
DESIGNED BY
DRAWN BY
SCALE
1 : 500
SHEET NO
1
DATE

GENERAL PLAN OF GARAI RIVER BRIDGE

SIDE VIEW SCALE 1:500



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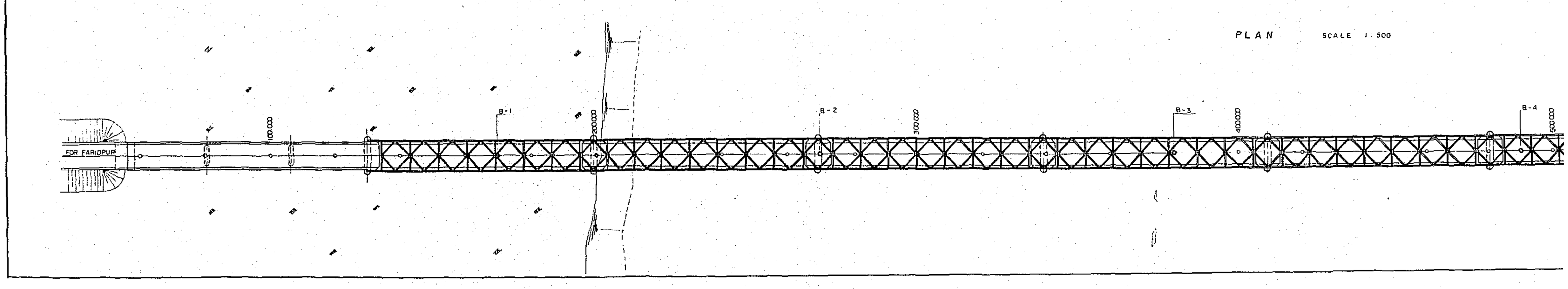
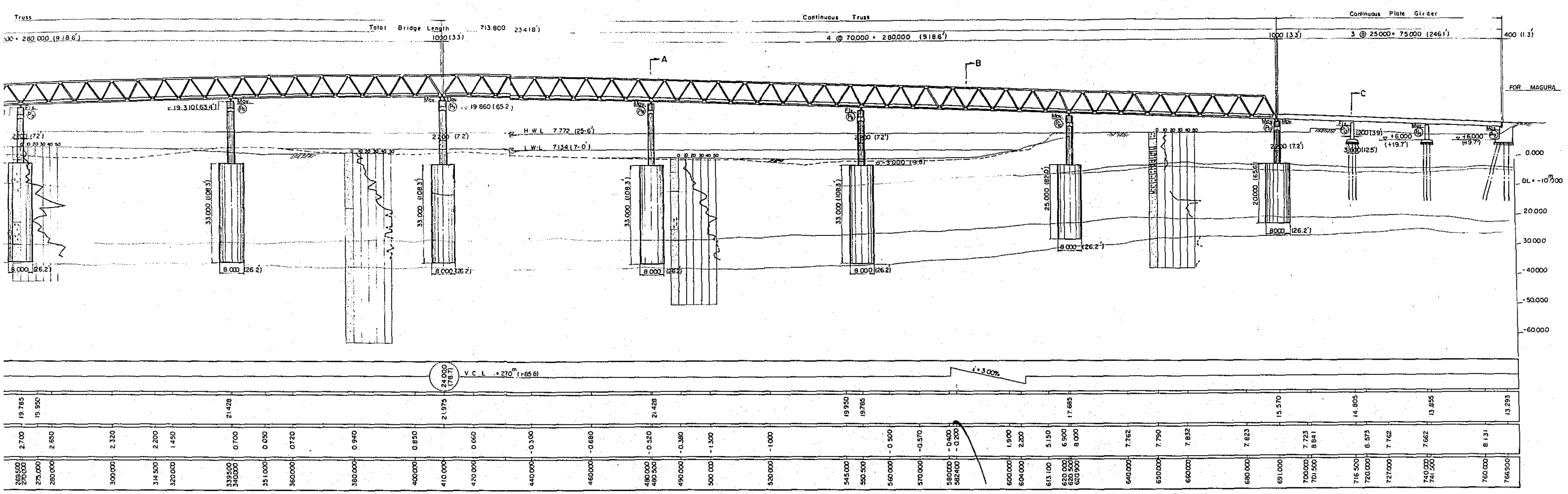
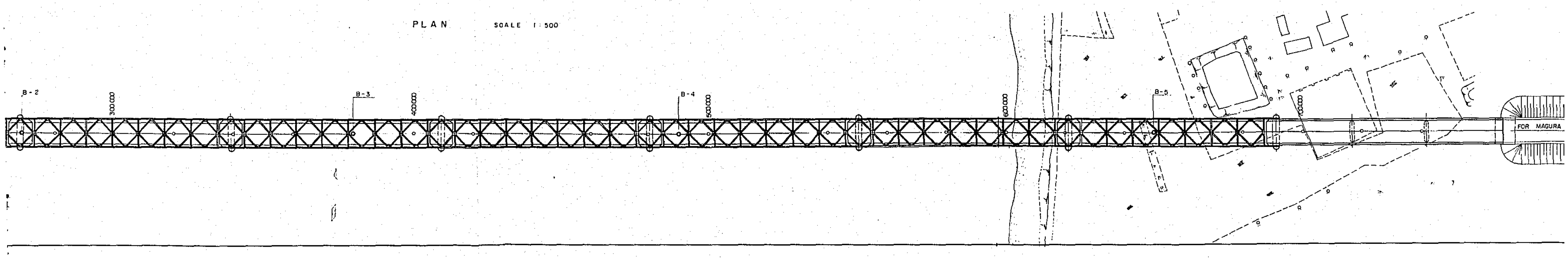


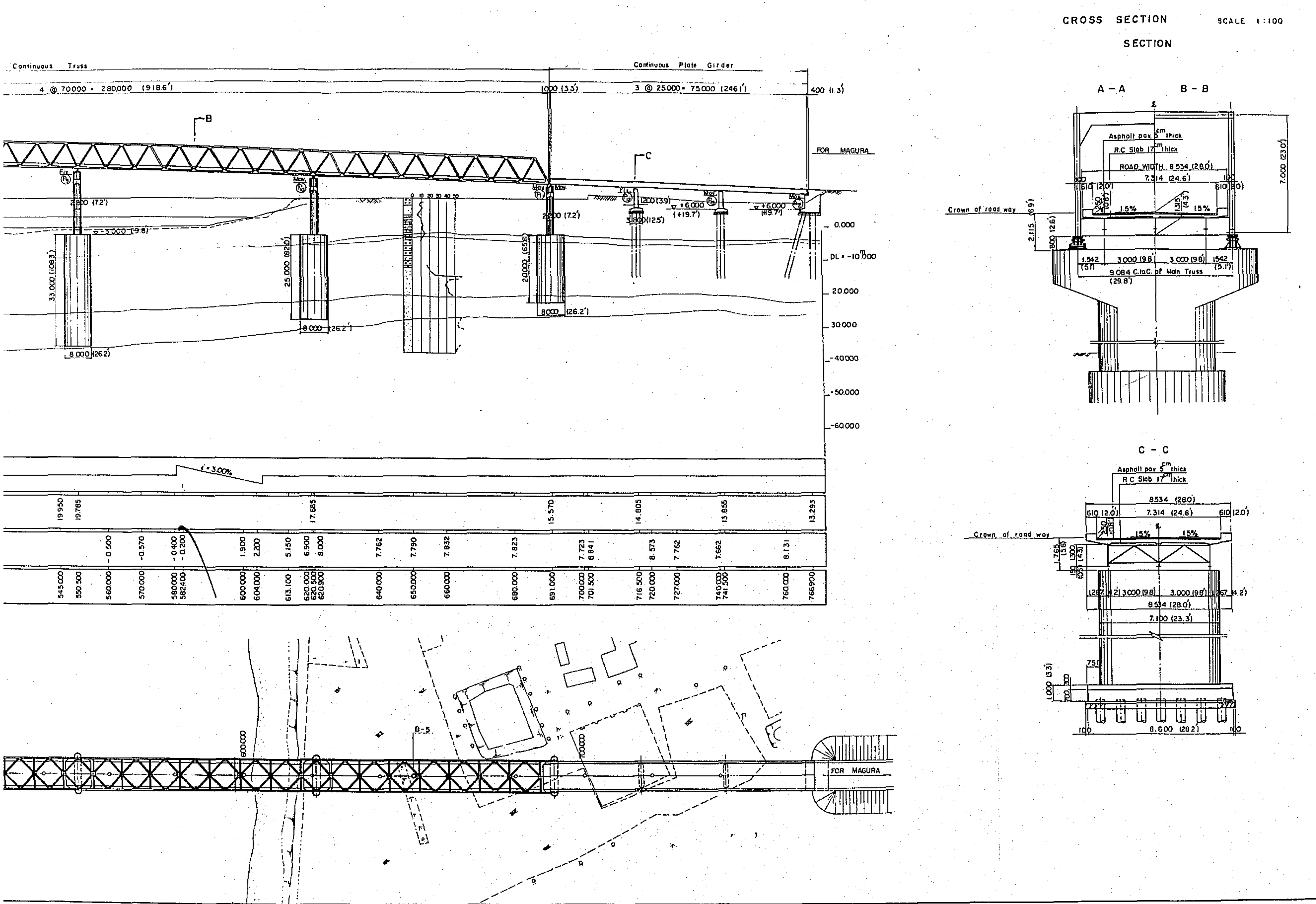
GENERAL PLAN OF GARAI RIVER BRIDGE

SIDE VIEW SCALE 1:500



PLAN SCALE 1:500





OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN

PACIFIC CONSULTANTS K.K, TOKYO JAPAN

EAST PAKISTAN

GARAI RIVER BRIDGE PROJECT

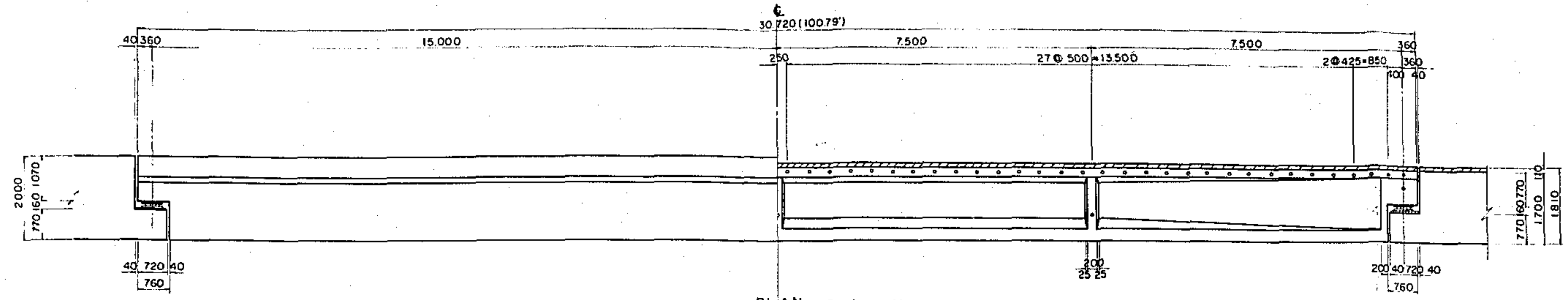
TYPICAL DETAIL STEEL BRIDGE

GENERAL VIEW

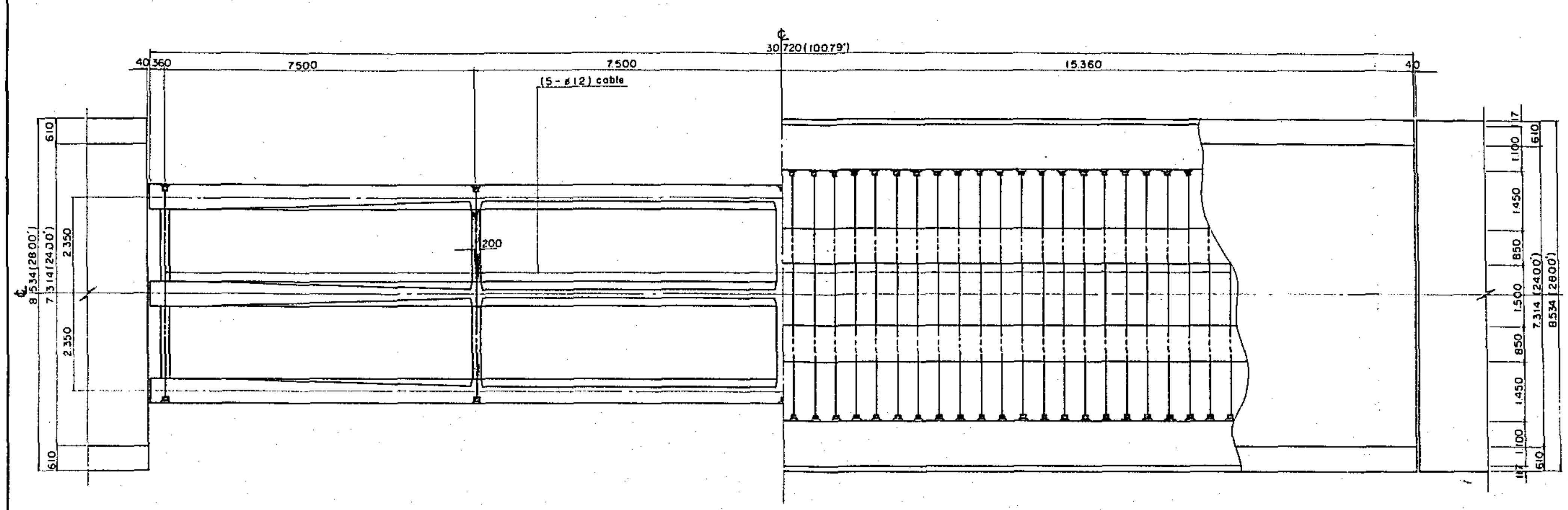
CHIEF ENG.	SCALE. 1:100 1:500
CHECKED BY	SHEET NO. 1.
DESIGNED BY	
DRAWN BY	DATE.

SIDE VIEW Scale = 1:60

OUTSIDE GIRDER — INSIDE GIRDER



PLAN Scale 1:60

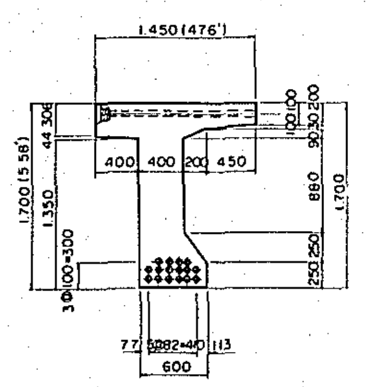


MATERIAL STRENGTH

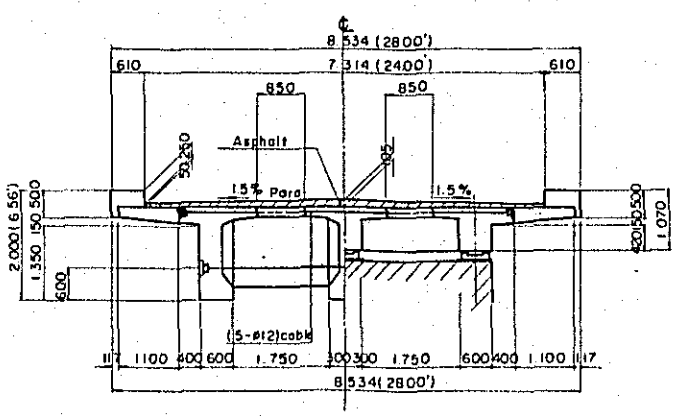
CONCRETE	PRESTRESSED CONCRETE SUSPENDED CANTILEVER GIRDER
COMPRESSIVE STRENGTH AT 28 DAYS	350 kg/cm²
ALLOWABLE BENDING COMPRESSIVE STRESS	115
ALLOWABLE BENDING TENSILE STRESS	-13.5
COMPRESSIVE STRENGTH AT TIME INITIAL STRESS	300
MAXIMOM SIZE OF GRAVEL	25 mm

PC WIRE	PRESTRESSING -WIRE ϕ 7 mm
ULTIMATE TENSILE STRENGTH	155 kg/mm²
YIELD POINT STRESS	135
ALLOWABLE TENSILE STRESS UNDER DESIGN LOAD	93
ALLOWABLE TENSILE STRESS AT PRESTRESSING	121.5

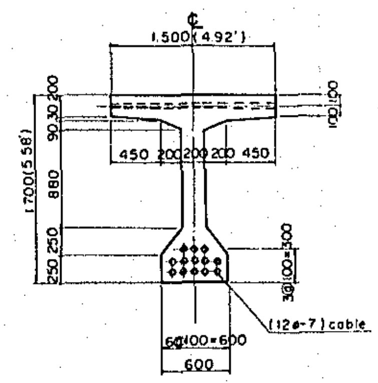
DETAIL SECTION OF OUTSIDE GIRDER Scale 1:30



CROSS SECTION Scale 1:60

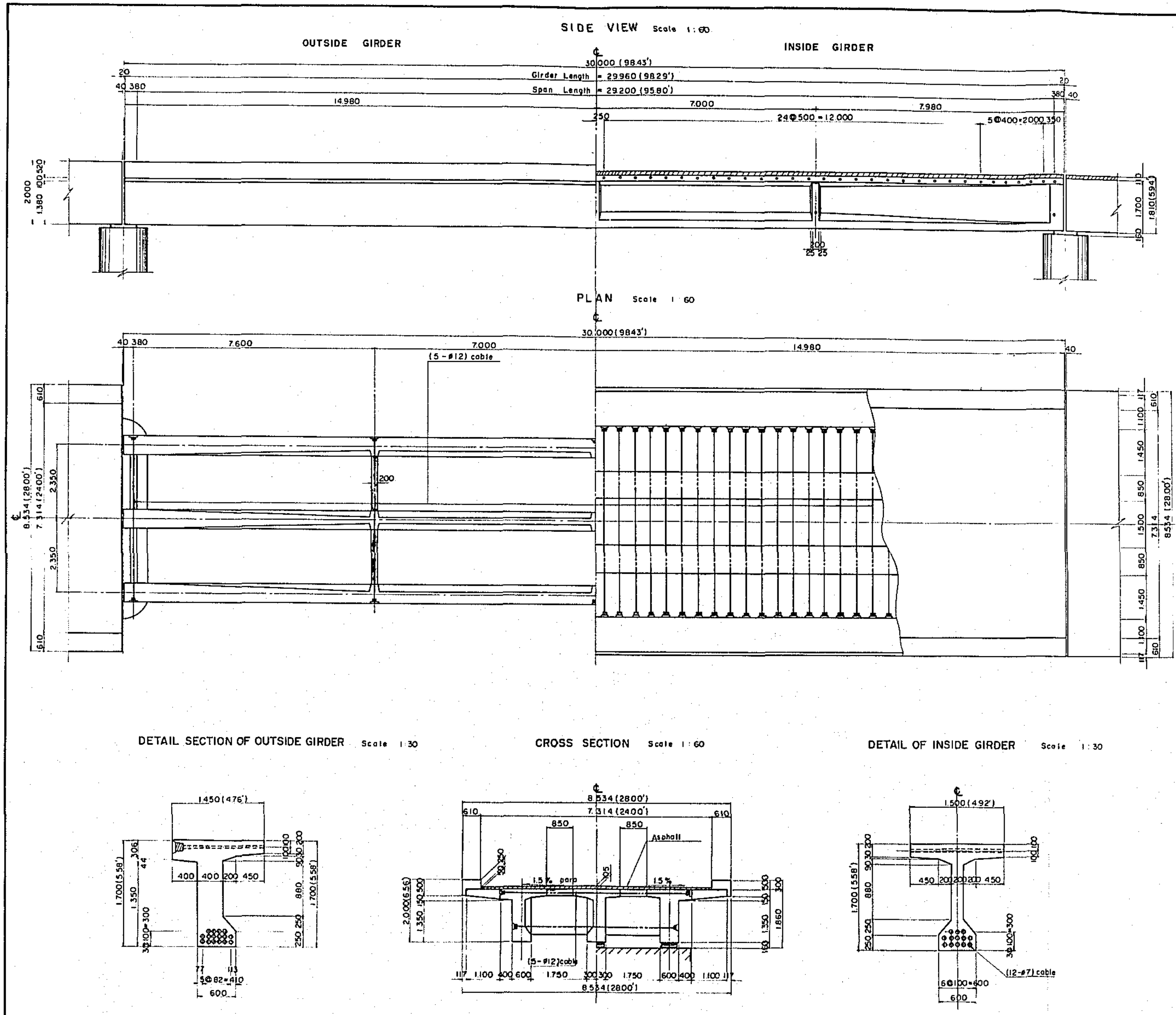


DETAIL SECTION OF INSIDE GIRDER Scale 1:30



OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN
PACIFIC CONSULTANTS K.K., TOKYO JAPAN
EAST PAKISTAN
GARAI RIVER BRIDGE PROJECT
TYPICAL DETAIL P.C. BRIDGE GENERAL VIEW

CHIEF ENG.	SCALE 1:60
CHECKED BY	SHEET NO. 5
DESIGNED BY	DATE
DRAWN BY	



MATERIAL STRENGTH

CONCRETE	PRESTRESSED CONCRETE SUSPENDED CANTILEVER GIRDER
COMPRESSIVE STRENGTH AT 28 DAYS	350 kg/cm²
ALLOWABLE BENDING CONPRESSIVE STRESS	115
ALLOWABLE BENDING TENSILE STRESS	- 13.5
COMPRESSIVE STRENGTH AT TIME INITIAL STRESS	300
MAXIMUM SIZE OF GRAVEL	25 mm

PC WIRE	PRESTRESSING - WIRE (#7) mm
ULTIMATE TENSILE STRENGTH	155 kg/mm²
YIELD POINT STRESS	135
ALLOWABLE TENSILE STRESS UNDER DESIGN LOAD	9[illegible]
ALLOWABLE TENSILE STRESS AT PRESTRESSING	1[illegible] 5

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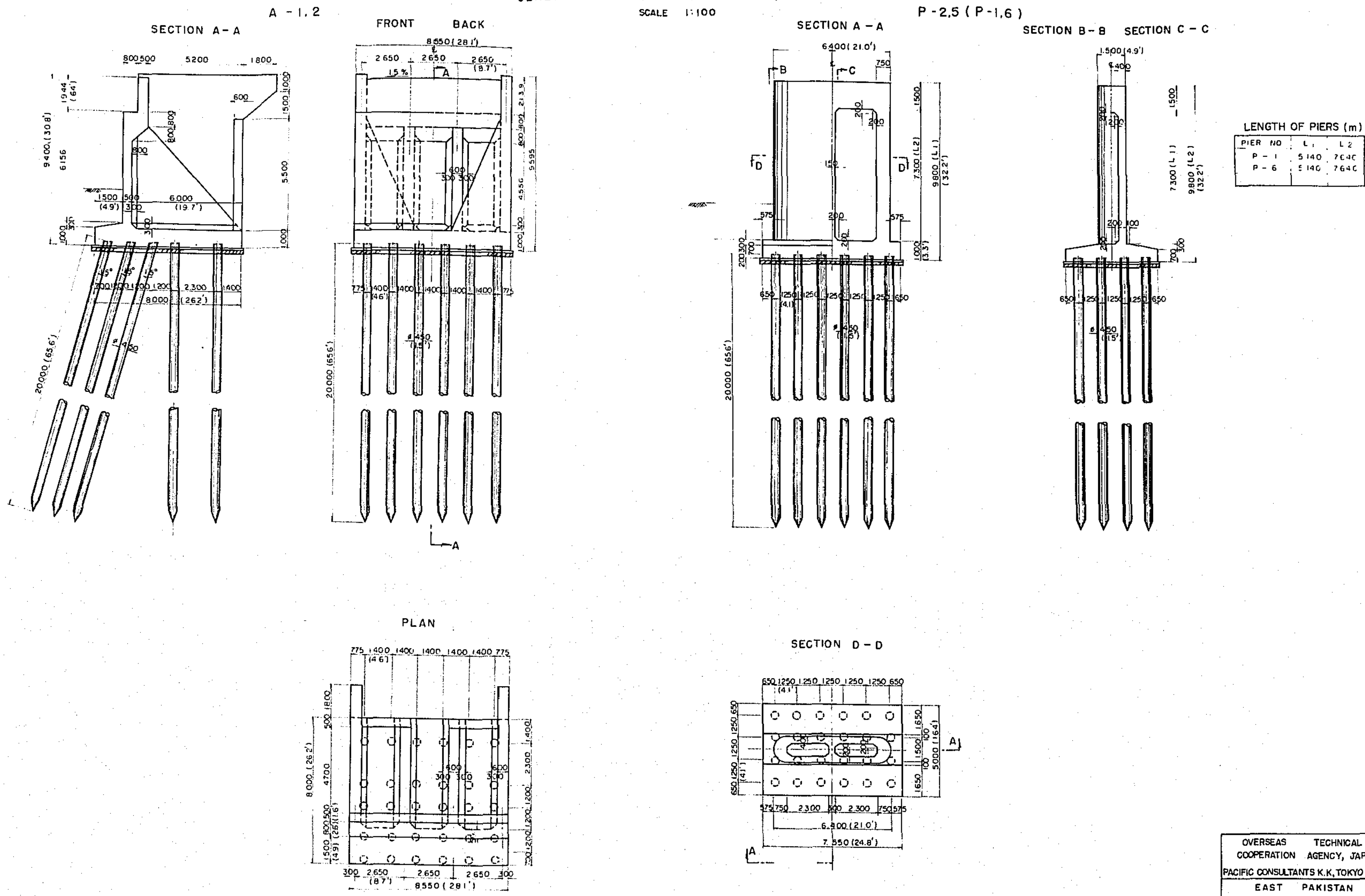
GARAI RIVER BRIDGE PROJECT
TYPICAL DETAIL P.C. BRIDGE GENERAL VIEW

CHIEF ENG	SCALE. 1:30 1:60
CHECKED BY	SHEET NO. 6
DESIGNED BY	DATE.
DRAWN BY	

GENERAL VIEW (FOR P.C. BRIDGE)

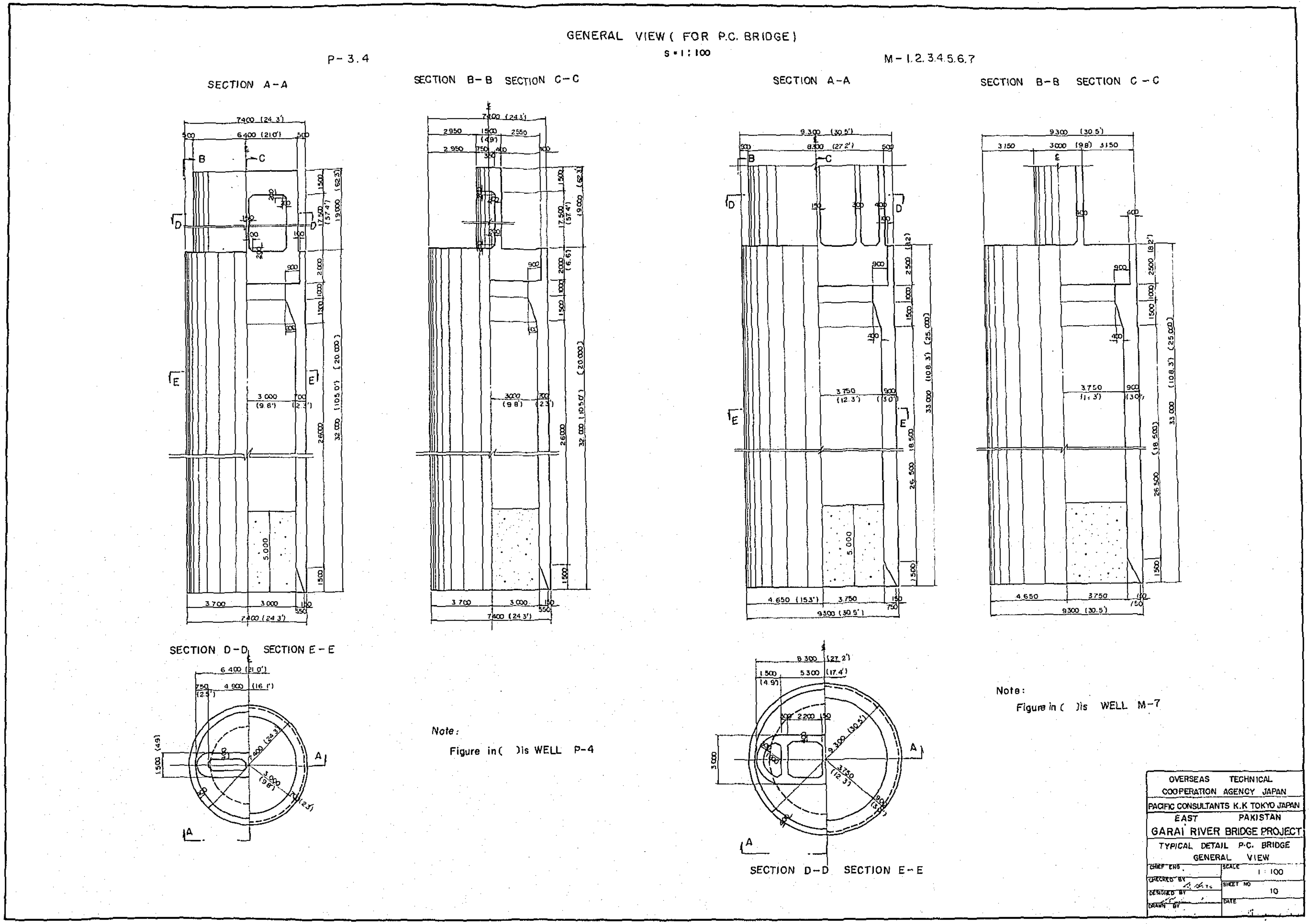
SCALE 1:100



LENGTH OF PIERS (m)

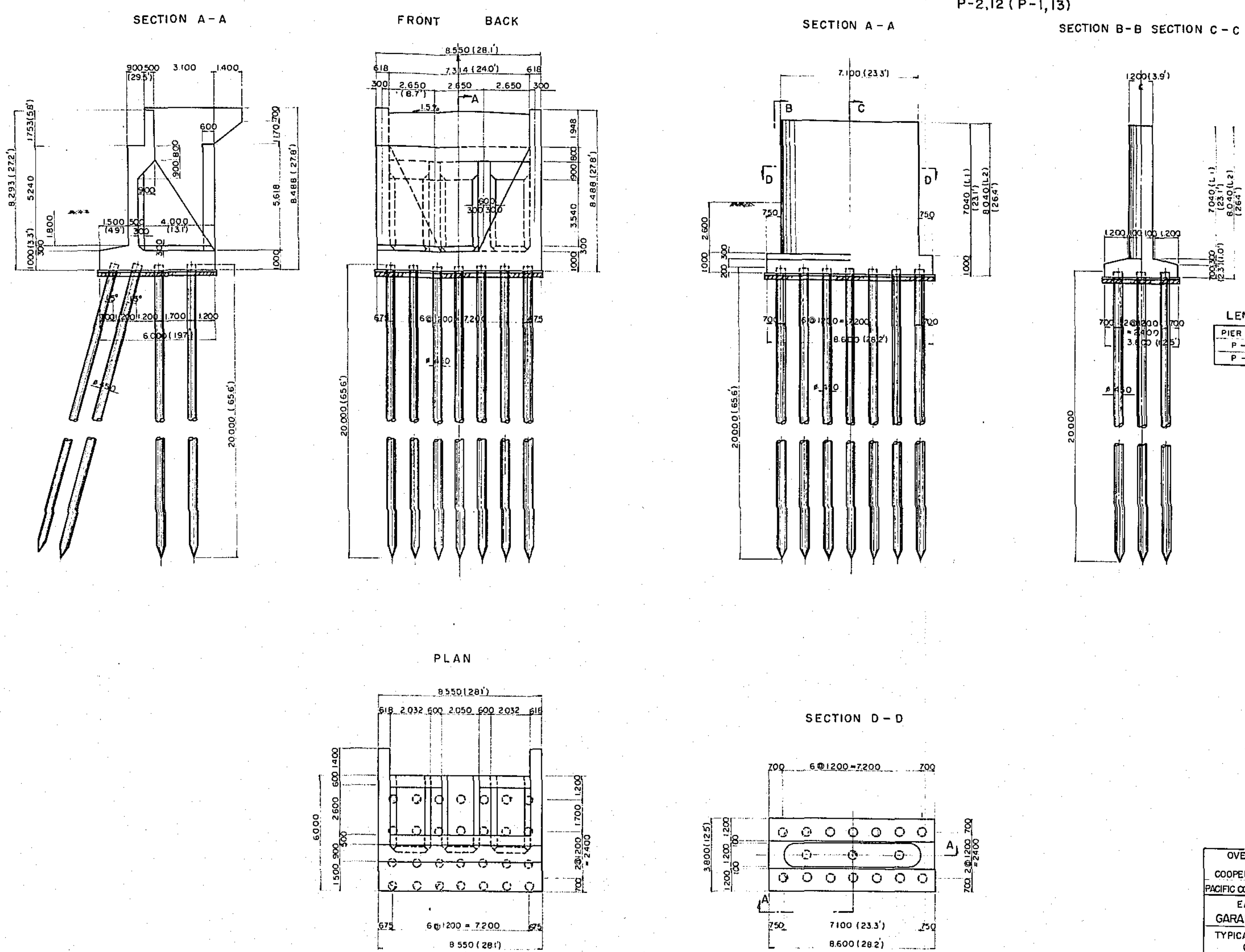
PIER NO	L1	L2
P - 1	5140	7640
P - 6	5140	7640

OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN	
PACIFIC CONSULTANTS K.K, TOKYO JAPAN	
EAST PAKISTAN	
GARAI RIVER BRIDGE PROJECT	
TYPICAL DETAIL P.C. BRIDGE GENERAL VIEW	
CHIEF ENG.	SCALE 1 : 100
CHECKED BY	SHEET NO. 9
DESIGNED BY	DATE
DRAWN BY	


GENERAL VIEW (FOR P.C. BRIDGE)
S = 1 : 100
P- 3.4
SECTION A-A
SECTION B-B SECTION C-C
M- 1.2.3.4.5.6.7
SECTION A-A
SECTION B-B SECTION C-C
SECTION D-D SECTION E-E
Note:
Figure in ( )is WELL P-4
SECTION D-D SECTION E-E
Note:
Figure in ( )is WELL M-7
OVERSEAS TECHNICAL COOPERATION AGENCY JAPAN
PACIFIC CONSULTANTS K.K TOKYO JAPAN
EAST PAKISTAN
GARAI RIVER BRIDGE PROJECT
TYPICAL DETAIL P.C. BRIDGE GENERAL VIEW
CHIEF ENG.
SCALE 1 : 100
CHECKED BY
SHEET NO 10
DESIGNED BY
DRAWN BY
DATE

GENERAL VIEW (FOR STEEL BRIDGE)

SCALE 1:100



LENGTH OF PIERS

PIER NO	L1	L2
P - 1	6.090	7090
P - 13	6.090	7090

OVERSEAS TECHNICAL COOPERATION AGENCY, JAPAN PACIFIC CONSULTANTS K.K. TOKYO JAPAN	
EAST PAKISTAN GARAI RIVER BRIDGE PROJECT	
TYPICAL DETAIL STEEL BRIDGE GENERAL VIEW	
CHIEF ENG.	SCALE 1 : 100
CHECKED BY	SHEET NO. 11
DESIGNED BY.	DATE.
DRAWN BY	

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