REPORT FOR CONSTRUCTION PROJECT
OF
KARNAPHULI RIVER BRIDGE AT CHITTAGONG
IN
EAST PAKISTAN

MARCH 1966

OVERSEAS TECHNICAL COOPERATION AGENCY

GOVERNMENT OF JAPAN

FOREWORD

The Government of Japan has, at the request of the Government of Pakistan, offered the consulting services in connection with the bridge construction programme in East Pakistan since 1960 and has already dispatched three survey missions for the reconnaissance of the road and irrigation programme, the reconnaissance of the bridge construction projects and the survey of the Burhiganga river bridge project at Dacca city.

The survey mission of this time has carried out the detailed field survey and preliminary design for the Karnaphuli River Bridge at Chittagong City which was recommended by the former reconnaissance mission in 1962, and has hereby submitted the present report.

The Overseas Technical Cooperation Agency, as an executive organ of the Government of Japan, has been performing such technical cooperation as the offer of consulting services to, dispatch of experts to and induction of technical trainees from developing countries.

Nothing would be more gratifying to us, if this report could be of any contribution to the promotion of the Karnaphuli River Bridge Construction Project at Chittagong City as well as to the furtherance of the amity, friendship and economic relations between Japan and Pakistan.

March 1966

SHIN-ICHI SHIBUSAWA
Director General,
Overseas Technical Cooperation Agency

TRANSMITTAL

December 28, 1965

Mr. Shinichi Shibusawa

Director General

Overseas Technical Cooperation Agency

Dear Sir,

I, as Leader of the Survey Mission for Karnaphuli River Bridge Construction Project, have the honour to present herewith the Report on the work assigned to and performed by the Mission.

The Mission, during its stay in the project area from January to April 1965, carried out such works as discussions with the Pakistani Authorities concerned, topographical survey and geological exploration at the proposed site, investigation on the river conditions, collection of meteorological data, and collection of informations necessary for the estimation of the traffic volume and construction cost as well.

In Japan, as the office work, the test on the soil samples obtained in the field investigations was performed by Tone Boring Co., Ltd. under the direction of the Mission. Further, the Construction Technique Institute Co. prepared the preliminary design based on the design specification made from the findings of the field investigations and the soil test. These results have been compiled into the present Report. Although due consideration was paid on the effect which would be caused by the flood in this project planning, it is yet desirable to confirm the flood effect by conducting in 1966 a further investigation on the river conditions in the flood season which could not unfortunately be conducted this year.

The city of Chittagong which is situated in the most inner part of Bengal Bay has developed, with the port as its center, into the second greatest city in East Pakistan with a population of 360,000. Chittagong Development Authority has, for the future development of the city, prepared the Master Plan for great Chittagong City which envisages a city expanding along the both sides of the Karnaphuri River by the rearrangements of the existing town and factory area in the western district of the River, the extension of the port, and the formation of industrial and residential areas in the eastern district of the River. Meanwhile, Asian Highway Route A-2 which links India, East Pakistan and Burma, is being contemplated to pass through the city.

Karnaphuli River Bridge will be a prerequisite to the realization of the above greater city planning, and will further play an important role in the Asian Highway Project. After the completion of the proposed bridge, the daily traffic volume in 1991 is estimated at more than 18,000 vehicles. As the result of the investigation, the Mission has planned to construct at Chaktai site, north-eastern district of the present city, a 2-lane permanent bridge with 26 feet roadway and 6 feet sidewalk on each side. From the viewpoint of navigation, economy and appearance, the proposed bridge, with an overall length 3,520 feet, will be of structure type with continuous steel truss for the main span and composite girder or reinforced concrete girder for the side span. Its total construction cost is roughly estimated at $4.6 million which, in the case of a toll bridge, could be refunded solely from its revenue in as short a period as 14 years after its opening. Judging from a financial standpoint, therefore, the construction project of this bridge is considered to be quite feasible.

It would be my utmost pleasure if this Report could serve the purpose of earlier realization of the proposed bridge construction project.

During its field survey, the Mission maintained close contact with the Provincial Government of East Pakistan, East Pakistan Industrial Development Corporation, Chittagong Municipality, Chittagong Development Authority, Chittagong Port Trust, Inland Water Transportation Authority, Chittagong Division of Communication and Building Department, Pakistan Eastern Railway, and the Consulate-General of Japan at Dacca.

I avail myself of this opportunity to express my deepest gratitude to the above-mentioned competent authorities and the organizations concerned for their invaluable cooperation and assistance.

With my renewed and heart-felt appreciation of your invaried support and encouragement, I remain, Dear Sir,

Yours very sincerely,

K. Omiya

Katsumi Ohmiya

Leader of Japanese Survey Mission for

the Karnaphuli River Bridge Construction

Project, Chittagong, East Pakistan

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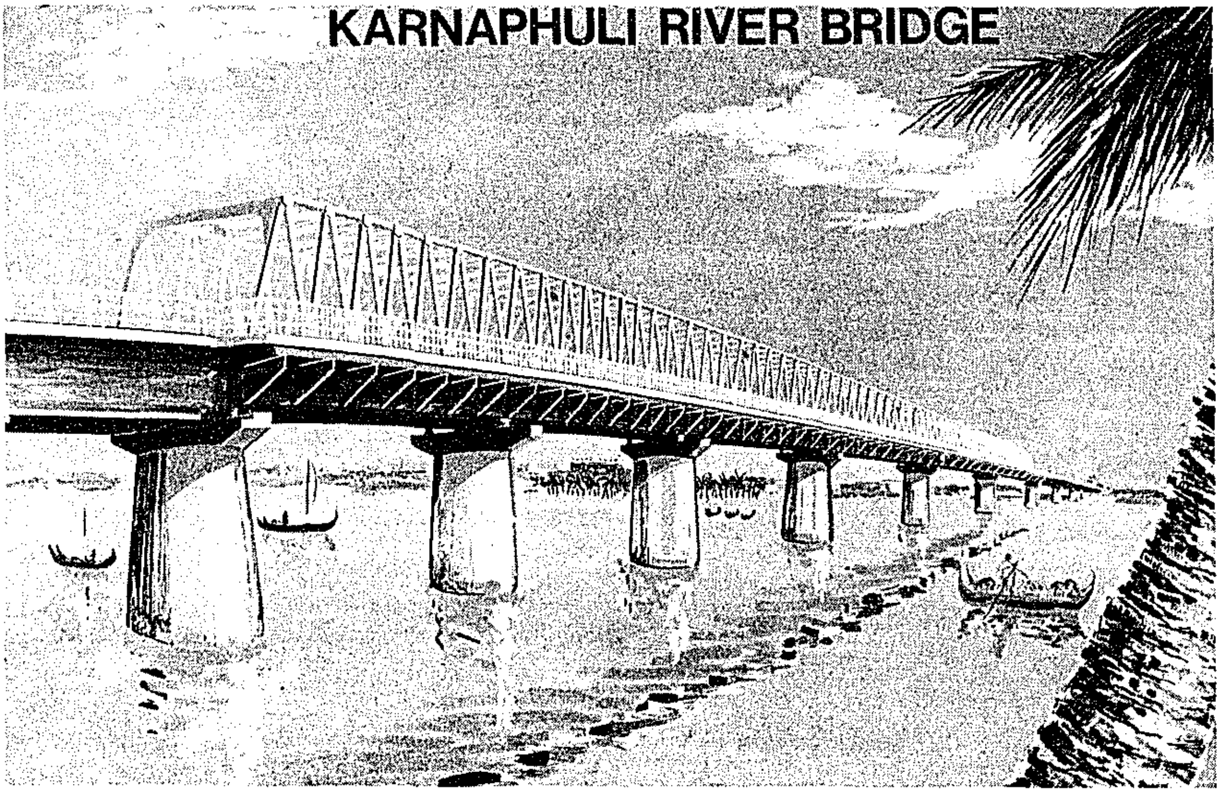


Karnaphuli River Bridge

Length	:		3, 520 ft. 4 in.
Width	:	Roadway	26 ft.
		Sidewalk	12 ft.
Type of Main Span:		3-span Continuous Steel Truss	

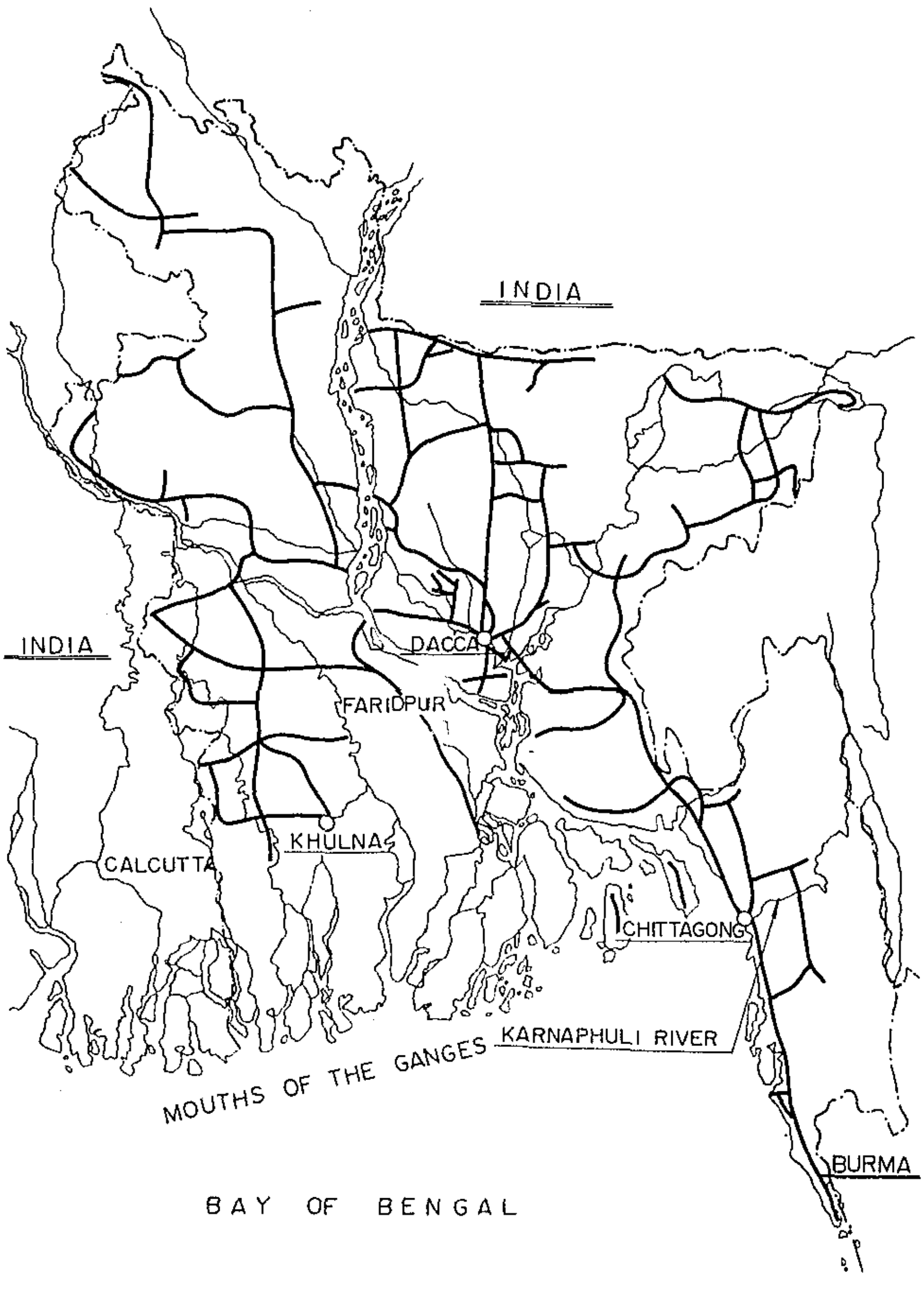


FIG - 1 LOCATION MAP OF EAST PAKISTAN

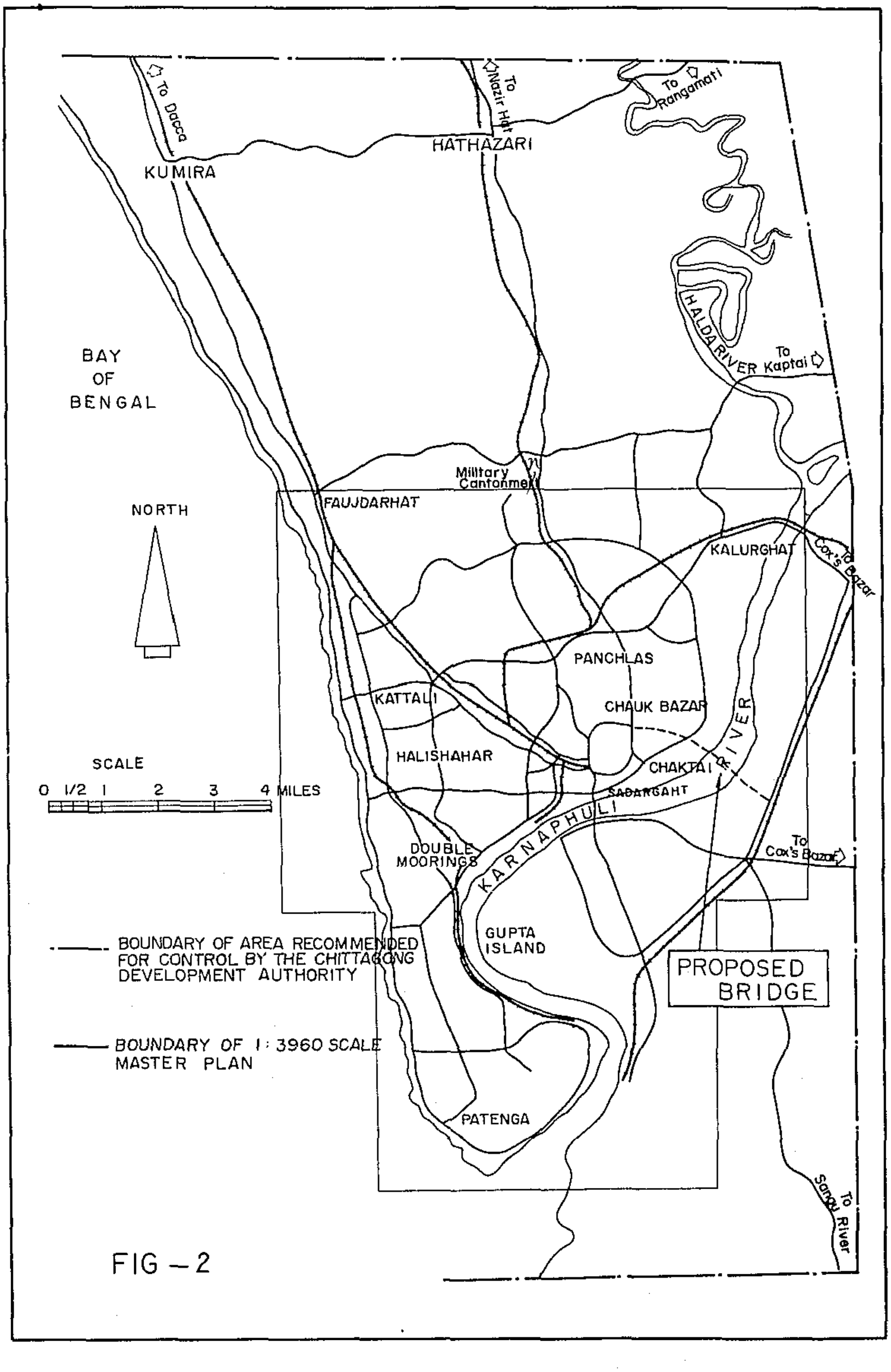

To Dacca
To Nazir Hat
To Rangamati
HATHAZARI
KUMIRA
BAY
OF
BENGAL
HALDA RIVER
To Kaptai
NORTH
Military Cantonment
FAUJDARHAT
KALURGHAT
To Cox's Bazar
PANCHLAS
KATTALI
CHAUK BAZAR
SCALE
0 1/2 1 2 3 4 MILES
HALISHAHAR
CHAKTAI
RIVER
SADARGAHT
KARNAPHULI
DOUBLE MOORINGS
To Cox's Bazar
GUPTA ISLAND
BOUNDARY OF AREA RECOMMENDED FOR CONTROL BY THE CHITTAGONG DEVELOPMENT AUTHORITY
PROPOSED BRIDGE
BOUNDARY OF 1: 3960 SCALE MASTER PLAN
PATENGA
To Sangu River

FIG — 2

CHAPTER I

GENERAL DESCRIPTION

1. Need for the Bridge

The Government of Pakistan has been showing the positive intension to construct a large industrial zone in the East Pakistan Province centering the two big cities of Dacca and Chittagong.

Chittagong is the city developed on the hilly hinterland along the west area of the lower reach of the Karnaphuli. The population is 360,000 as of 1961, and the town area is crowded with the rows of houses along narrow streets, generally forming a bazaar of the old time. The modernization would therefore be hardly effective unless a new city planning should be forced into operation in a new area, making the houses dispersed and decentralized at the time. In 1960, the Government of East Pakistan Province established the Chittagong Development Authority, which worked out the master plan for the Great Chittagong City. In this plan, the designated industrial zone is situated along the west bank of the Karnaphuli, where an ironworks and other large-scale manufacturing plants have already been put under construction one after another. It might be said that Chittagong City assuredly marked its first step to its industrialization.

Almost all export and import cargoes of this province in fact pass through the port of Chittagong: In 1961 the handling cargo amounted to a total of 4.5 million tons. Keeping pace with the promoted industrialization of Chittagong City and East Pakistan Province, the cargo of export and import has been on a rapid increase, and it is expected that by 1991 the cargo would exceed 10 million tons. The port facilities have already become tight, and it is alleged that it would forcedly take several fortnights for ocean-going vessels to load and unload. Under the circumstances, this only one oversea trade port of the Province should necessarily expand its facilities. According to the plan of Chittagong Port Trust, new jetties are to be constructed along the east bank of the river in the new 5-year plan. Further in the master plan of the C.D.A., the areas for warehouses, factories and housing are separately to be developed in the adjoining area of the port facilities.

The east and west bank areas of the river are at present only connected with the privately-operated ferryboat and the one-lane road bridge which is made use of the Kalurghat railway bridge located 7 miles in the upper stream from Chittagong City. Nothing but the construction of the road network can only inspire the Great Chittagong with vitality so as to secure an automobile traffic. The C.D.A. has already started its activity by bringing into operation its master plan. At this stage it is convinced the best way to set to the bridge construction as soon as possible, enabling to materialize the city planning and to facilitate the city modernization as well. The east and west bank areas would be closely tied up with a bridge. If going by way of the existing Kalurghat Bridge, it took about 60 minutes to go 15 miles including the waiting time due to the 1-lane road. On the other hand, the new bridge will make it possible to shorten the distance down to 3 miles and the required time down to 7 minutes. The driving expenses and the toll would be greatly saved. With regard to the pedestrian, they can go by bus in less than 10 minutes from the central part of the city to the opposite bank, meanwhile it took more than one hour by ferry boat service. Further, this bridge always guarantees a smooth passage in any weather and at any time. This function could not be attained at all with the Kalurghat Bridge, ferry boat or a pontoon bridge.

2. Outline of the Survey Mission

The Government of Japan has entrusted the execution of the survey for the Karnaphuli River Bridge Construction Project at Chittagong City to the Overseas Technical Cooperation Agency. The Agency dispatched a survey mission which stayed in Pakistan from January 14 to April 2, 1965.

The members of the mission are as follows:-

Katsumi OMIYA, Leader of the mission,	Staff, Kawasaki Dockyard Co.
Yukio MAEDA	Staff, Sakurada Iron Works Co.
Toshio WADA	Staff, Construction Technique Institute Co.
Naoe NAKAMURA	Staff, Japan Highway Public Corporation
Yumio ISHII	Staff, Construction Technique Institute Co.
Masashi KUNIHIRO	Staff, Kawasaki Dockyard Co.
Yasuo FUKUI	Staff, Tone Boring Co.
Torao GENKE	Staff, Tone Boring Co.
Toshimitsu OZEKI	Staff, Tone Boring Co.
Junichi HAYASHI	Staff, Tone Boring Co.

The soil test was performed by Tone Boring Co., Ltd., and the preliminary design was prepared by Construction Technique Institute Co.

3. Site Investigation

The outlined works of the investigation in Pakistan are as listed below:-

(1) Selection of the location of the link.

(2) Setting-up of the center line of bridge in field, and determination of bench-marks.

(3) Topographic surveying and sounding.

(4) Subsurface exploration.

(5) Collection of meteorological data.

(6) Collection of informations for probable traffic increase.

The location of the bridge site would not only influence the construction cost, but also it may become a dominating factor for the development of the neighbouring area. According to the C.D.A.'s Master Plan, Sadarghat is best situated connecting the opposite bank area with the central part of Chittagong City in the shortest distance. Contrarily, Sadarghat is to be included in the future expansion plan of Chittagong port facilities, and it is undersirable to locate a common-type bridge in the lower streams from Sadarghat included.

Therefore, it was decided necessary to make the study on three draft plans including the tunnel plan, so that an open water basin can be left for the port. The demerits of the plans which put the link at Sadarghat are as follows: -

(1) In the tunnel plan, the total length reaches 10,000 feet including the access roads, costing about 5 times as much as the bridge construction, and the cost for maintenance and repair would need quite a large amount of money.

(2) For the suspension bridge should be required a 150 feet clearance to enable 9,000 tons vessels to navigate under it. This should require long access roads. On the other hand, the crowded houses on the route would cause difficulties in land acquisition. The construction cost will take about the same amount as the tunnel plan.

(3) The movable bridge will put restrictions on both the vessel navigation and road traffic.

At the joint-conference with the authorities concerned, all the plans were given up, and it was agreed upon that a new link should be selected near to Chaktai and upstreams from the point where the accident had took place with the ships drifted by the cyclone. In accordance with the decision, a desk study and site survey were carried out to select the suitable site at the location 1 mile upstreams from Chaktai.

4. Superstructure

It is economical in principle that the bridge length should be made out to the shortest extent in total. The abutment on the left bank was designed to be put just inside of the embankment. On the right bank, the abutment was determined to be placed a little inside of the shoal, which is in the Master Plan expected to be reclaimed for an effective land use. As the result, the bridge length is made to be 3,520 feet 4 inches in total.

The expected traffic volumes would be composed of the transferred traffic from the railway bridge and ferryboat, the generated traffic from the development of the new residential area, factory area and port facilities, and the long-distance traffic caused from the opening of the Asian Highway. The peak-time traffic is assumed to be 10% of the daily traffic. The estimated traffic volumes are as follows:-

	Daily traffic	Hourly traffic
1971	4, 097 vehicles	410 vehicles
1981	11, 107	1, 110
1991	18, 397	1, 840

It is considered that a 2-lane bridge would be enough serviceable approximately by 1990. The width of the bridge is made out as follows: -

(Sidewalk 6 feet + Roadway 13 feet) x 2 = Effective width 38 feet

The maximum span length is made to be 250 feet because of being the most economical and causing no trouble for the ship navigation. The clearance above the mean water level is decided to be 40 feet, which enables the ships of 500 ton-class to navigate under the bridge.

The steel truss is the most suitable structure for the main span. Since the foundation is reliable, either a cantilever type or a continuous type can be constructed safely enough. However, the cantilever bridge is rather expensive and more complicate in structure. Accordingly, the continuous truss bridge is accepted. The side span would be little restricted by the navigation limit, so can be selected to construct short-span bridges. The comparative study was made for two types of bridge: a combined type bridge with long-span truss and short-span girder which is hereinafter called Type A, and the other all-span truss bridge which is hereinafter referred to as Type B.

5. Substructure

The soil at the bridge site belongs to the alluvium. The strata consists of the alternation of sand and silt, and the N-value of the standard penetration test is some 30 at the depth of 10m and reaches 40 at the depth of 20m. Except for a small part of the surface soil, there is nothing of the soil which may cause the secondary consolidation settlement. It is therefore possible to use any type of foundation for the bridge, provided it is borne at the sand stratum.

The depth of the probable scouring was estimated as 33 feet in accordance with the Lacy's empirical formula and Laursen's experimental one. The scouring causes no trouble about the vertical bearing power of foundations but fatally decrease the horizontal bearing power.

The pile foundation would maintain a small horizontal bearing power. This is a drawback to the foundation for the long span. Consequently, it is decided that the foundation works for the continuous truss should be made with the wells, and only those for the concrete girder of simple support made with the steel pipe-piles. The well foundations are designed to be the concrete colomn for the upper part in order not to much interfere with the river flow.

After all, in case of Type A bridge of which superstructure is a combined type, the steel pipe-pile foundations are carried out for the both abutments and 6 piers of the side spans, leaving the other 15 piers to the well foundation work. For Type B, all-span truss bridge, the foundation works are conducted by the well method all for 2 abutments and 13 piers.

6. Construction Cost

The construction cost is estimated for each of the 2-lane bridges, Type A and Type B, as follows: -

	Type A	Type B
Field Cost	US$4, 150, 000	US$4, 221, 200
Investigations	US$ 83, 000	US$ 83, 000
Engineering	US$ 290, 000	US$ 296, 000
Other	US$ 79, 000	US$ 84, 000
Total	US$4, 602, 600	US$4, 684, 200

The field cost is divided into two: one payable in the foreign currency and the other in the Pakistani currency.

	Type A	Type B
Foreign Currency	US$2,536,700	US$2,697,300
Pakistani Currency	US$1,613,900	US$1,523,900
Total	US$4,150,600	US$4,221,200

The construction cost is also estimated for the 4-lane bridge, which has an effective width of 60 feet and requires a construction period of 4 years. The estimated cost is US$6,861,000 for Type A and US$6,981,300 for Type B.

Recommendable is Type A that can use as much Pakistani materials as possible and requires the less construction cost.

CHAPTER II

CONCLUSION AND RECOMMENDATION

1. Conclusion

(1) The construction of a bridge over the Karnaphuli is assuredly feasible in engineering aspect. From the view point of navigation, economy and appearance, it is advantageous to adopt continuous steel trusses for the main span, and composite girders or reinforced concrete girders for the side span. The 2-lane bridge is economically preferable to the 4-lane bridge. The construction of the 2-lane bridge would require a cost of 4.6 million dollars and a work period of 3 years.

(2) The toll rate might be reasonably decided as follows:-

		(Kalurghat Bridge)
Truck & Bus	US ¢ 30	US ¢ 105
Passenger car	US ¢ 20	US ¢ 53
Auto Rickshow	US ¢ 10	US ¢ 21
Pedestrian	US ¢ 2	Not permitted

The whole construction cost of the 2-lane bridge is calculated to be repayable with the toll income in a comparatively short time namely 14 years from the completion on condition that the interest rate would be 6% per annum. On the other hand, the 4-lane bridge would take 21 years to repay the loan.

(3) The estimated traffic volume will have come to 18,400 vehicles a day by the end of 1991, this being the maximum limit for 2-lane bridge. Presently other new facilities will be demanded to link the both sides of Karnaphuli River. On that occasion, the construction of new facilities will be easily achieved with the reserved fund from the toll which may be collected for some time after the repayment is completed.

2. Recommendation

(1) The Government of Pakistan should recognize that a permanent bridge across the Karnaphuli is indispensable to assure the modern traffic for the development of the Great Chittagong City, and should immediately take such actual actions as the financial arrangement and the joint conference of engineers of Pakistan and Japan.

(2) The rainy season survey of the river must be carried out in 1966, in order to study the flooding of the river water, the change in the watercourse and the extent of scouring at the Kalurghat Bridge, and to obtain the engineering data necessary for the final design and construction planning.

Further, it is desirable, before starting the final design, to carry out the hydraulic model test using the movable bed materials in order to study the influences of the bridge construction upon the river hydraulics as well as the protection works for them.

(3) The preliminary design in this report is only confined to the bridge and the just adjoining portions of the approach road, and the other portions are left to be hereafter planned and designed by engineers of the CDA. It is recommendable to take the following points into consideration: On the right bank, the embankment of the approach road would be intruded for the distance of 2000 feet into the lowland which would be submerged in water at the time of high tide, and the banking would reach the height of 12-22 feet. Therefore, the soft clayey layer of the surface soil should be replaced by sand in order to increase the bearing power and to prevent the side-slope collapse and settlement due to consolidation. The flooded water in the upstream area from the embankment would have to be flown out at the ebb tide. For the purpose of draining water along the embankment, a channel should be excavated a certain space away and in parallel with the embankment.

(4) The CDA has so far marked a remarkable achievements in the development of Chittagong City, having a good engineering staff. However, the construction of a long bridge would require the higher engineering technique and richer experiences. So, the training of the engineers should be included in the project, and it may be de-

sirable for them to be sent abroad as the trainee, for example, in the Colombo Plan.

(5) The existing master plan is unmatured for the development plan of the east bank area, and the bridge location is selected at the different place. It would be necessary to make a new master plan featuring of a close organic unity of the bridge and the city planning. In the new master plan, the road plan should be decided to satisfy both city traffic and through traffic.

(6) In this design, it is presumed the main flow of the Karnaphuli River runs a little to the left from the center line. Judging from the existing record of the past running of the watercourse, the present watercourse would not probably be maintained. Such a movement of the watercourse as cause the erosion on the river sides near the bridge abutments should give a fatal damage to the bridge.

The river training works are now in process based on the model test of the Hydraulic Research Station. Near the proposed bridge site, the training works are to be constructed on both banks so as to stabilize the river course. All along the both banks through the upper and lower streams, the training works should be constructed in order not only to secure the safety of bridge but to maintain the water depth at the port area and to prevent the flooding all over the river side area.

(7) The construction site of the bridge was decided in the upper stream from the port facility block so that the bridge might be protected against the collision with the ships that fearfully may be drifted by cyclones. But, this is not the perfect safeguard against the possible collision, and the mooring buoys should be well equipped with and at the same time the careful weather observation should be continued.

CHAPTER III

LOCATION OF BRIDGE

1. Sadarghat Site

In 1962, the preliminary survey for the construction of a bridge over the Karnaphuli had been carried out by the Japanese Bridge Survey Mission led by Dr. S. Inagaki. According to the reconnaissance report, a draft plan was made up for the construction of the bridge across the Karnaphuli at Sadarghat south of the central part of Chittagong City (See Fig. 2). After being informed about the Master Plan of Chittagong Development Authority and making an inspection of the site, the Mission of this time also confirmed that Sadarghat was located very much suitable for a bridge construction in the scope of city planning, existing river regime and future prospect over the development of the left bank area.

On the other hand, with a view of making the field investigation based on the Sadarghat plan, the Mission made discussions with the Pakistani authorities concerned, and it came to know that the bridge location should be restudied accordingly as the situation had changed since 1962.

2. Views of Authorities Concerned.

2. 1. Chittagong Port Trust

Mr. S. Zaman, Chief Engineer C. P. T., and others.

The improvement plan of the Chittagong Port was outlined as follows: The 5-year plan were under deliberation for the expansion of the existing 9 jetties into 14 ones. According to it, new port facilities would be constructed all the way down the stream from Sadarghat included, where no bridge would therefore be able to be constructed. In 1962, a big cyclone hit East Pakistan. It was said reportedly that the wind velocity of that cyclone should be 110 - 130 miles per hour in the Chittagong District, causing a number of ships in anchorage, large or small, drifting to float. Some of them were in fact blown so far as up to Chaktai. Taken into consideration that a cyclone of the same scale might strike the district again in future, the bridge construction between Sadarghat and Chaktai should be made out to avoid the danger of ships' collision against the bridge. The substructure should be safeguarded by the protective measures installed around the piers, and the superstructure should hold the navigation clearance for the ocean-going vessels.

The C. P. T. maintained the conclusion that a tunnel should be preferred to a bridge. In case of the tunnel constructed, there were such a advantage that no restriction would be placed by the navigation channel and vessel size.

2.2 East Pakistan Industrial Development Corporation

Mr. H. Meding, Engineering Adviser and Chief Engineer, E. P. I. D. C.

As regards the plan for industrial development along the Karnaphuli River, it was made clear: (1) The total 5,550 feet landing facilities will be completed along the upper stream on the opposite bank of Sadarghat. Ships which will come along these landing facilities in future will be as large as 8,000 - 9,000 tons. It is requested, therefore, that no bridge might be constructed in the proposed areas of these facilities, or that a sufficient clearance should be left under the bridge if the bridge would be constructed in these areas. (2) In back of these facilities, a large factory site will be prepared, which would necessarily require the construction of a bridge across the river.

2.3. Inland Water Transportation Authority.

Captain Shafe, I. W. T. A. Chittagong Office

It turned out clear that no larger ship than those navigating at present would be expected in the upper stream from Chaktai. It was requested that the clearance of the new bridge would be a little larger than that of the existing Kalurghat railway bridge.

2.4. Communication and Building Department

Mr. H. Rahman, Superintending Engineer, C. & B. Chittagong Circle.

The definite construction project of the trunk road network in the district of Chittagong, and the Asian Highway A-2 Route through Chittagong had not been made out yet. He was in the opinion that (1) if the bridge were linked to the central part of the city, it would not be suitable for a by-pass itself, meanwhile if it passed across the upper stream from Chaktai connecting with the outer loop road line, it should be most desirable for the A-2 Route, and (2) the C. & B. consider the A-2 Route to be a rather future problem.

2.5. Chittagong Development Authority.

From the standpoint of the city planning of Chittagong, the bridge construction site is desirable to be near the central part of city. Since the bridge construction, however, is to be carried out for the ultimate purpose of developing the Great Chittagong City including the opposite bank area, the bridge location might be chosen somewhere along the upper stream from Sadarghat. If the location should be too far up the river, the significance of the bridge would be lost in its original intention.

3. Determination of Bridge Location

On 22nd January, a joint-conference was held at the conference room of the C.D.A. under the auspice of chairman of the C.D.A. with the following attendants:

C.D.A.	Chairman, Hasan Nawab
	Superintending Engineer A.M.Z.H. Mazumder
	Executive Engineer A.A.M.Z. Hussain
P.E. Railway	Chief Engineer A. Chowdhury
	Deputy Chief Engineer A. Ghafur
E.P.I.D.C.	Chief Engineer H. Meding
C.P.T.	Chief Engineer S. ZAMAN
Chittagong Municipality	Chairman Lt. Col. Z. Hasan
Japanese Survey Mission Members	

In regard to the tunnel plan, the Mission stated its viewpoints as follows: -

(1) There are two methods to construct a tunnel, the shield method and sunken-tube method. The sunken-tube method is difficult to be executed at the Karnaphuli owing to the rapid stream of about 4 knots. In the shield method, the interior air pressure should be increased during excavation in order to prevent the water invasion; while the sufficient thickness of the riverbed soil is required to cover the tunnel for fear that the compressed air should not gush out. For such silt or sand stratum as seen at the Karnaphuli, the depth from the deepest riverbed to the top of the shield should be more than 50 feet. Therefore, if the maximum grade of the roadway is assumed to be 3%, the total length of tunnel would reach 10,000 feet (tunnel: 6,700 feet, approach road: 3,300 feet), or 2 times as long as that of a bridge.

(2) For a long tunnel, the ventilation should be equipped to send out the exhausted gas from automobiles.

(3) As the same design conditions are adopted in both bridge and tunnel plans, there is little difference in regard to the traffic capacity, while with regard to the construction cost, tunnel is nearly 5 times as much expensive as bridge. The estimated construction cost is $21,130,000 (tunnel: $18,350,000, ventilation equipment: $2,780,000).

(4) In maintaining the tunnel, accessories such as ventilation, illumination and drainage must be always operated, and a reserve generator is needed against the stoppage of electricity. While in case of the bridge, the essential maintenance is simply for illumination and painting.

Then, the Mission suggested three comparative locations of the bridge, and explained the relative merits of them as follows: -

Table 1. Comparison amomg Bridge Locations

In connection with;	Location		
	Between Kalurghat and Chaktaighat	Near Chaktaighat	Between Chaktaighat and Sadarghat
City Planning	C	B	A
Port Planning	A	B	C
Trunk Road Planning	A	B	C
Industrial Development	B	A	A
River Hydraulics	B	A	A
Subsurface Condition	B	A	A
Right of-way and Compensation			
Right Bank(City Side)	A	B	C
Left Bank	A	B	A
Construction Cost	B	A	B

Remark: A, B and C show respectively better, good and poor.

The chief engineer of the C. P. T. asserted: In the past a certain cyclone cut off the mooring rope to cause ships drifting up as far as Chaktai, and therefore, the bridge should be constructed upper the river from Chaktai in order to secure the safety of ships.

The chief engineer of the E. P. I. D. C. mentioned that the bridge was desirable to be constructed along the upper stream from Chaktai, because a new navigation channel would be made for ocean-going vessels (9, 000-ton class) to meet the development of the factory area on the right bank of Chaktai.

Finally, all conference attendants were unanimous in the opinion that the bridge location should be best suitable to be as near Chaktai as possible from the viewpoint of future development of the city.

According to the conclusion of the joint conference, the Survey Mission started a field survey. In conducting the survey, it is taken into account that the bridge could be located in the place to be easily connected to the road net of the Master Plan and to be utilized as a part of the Asian Highway A-2 Route. At the same time, attention was paid to that the bridge should be located in an advantageous position in the hydraulic point of view, besides being in the area requiring smaller compensation for the land expropriation.

The selected location shown in Fig. 2 were reported to the chairman of C. D. A., and the Mission immediately put into operation the surveying and boring works.

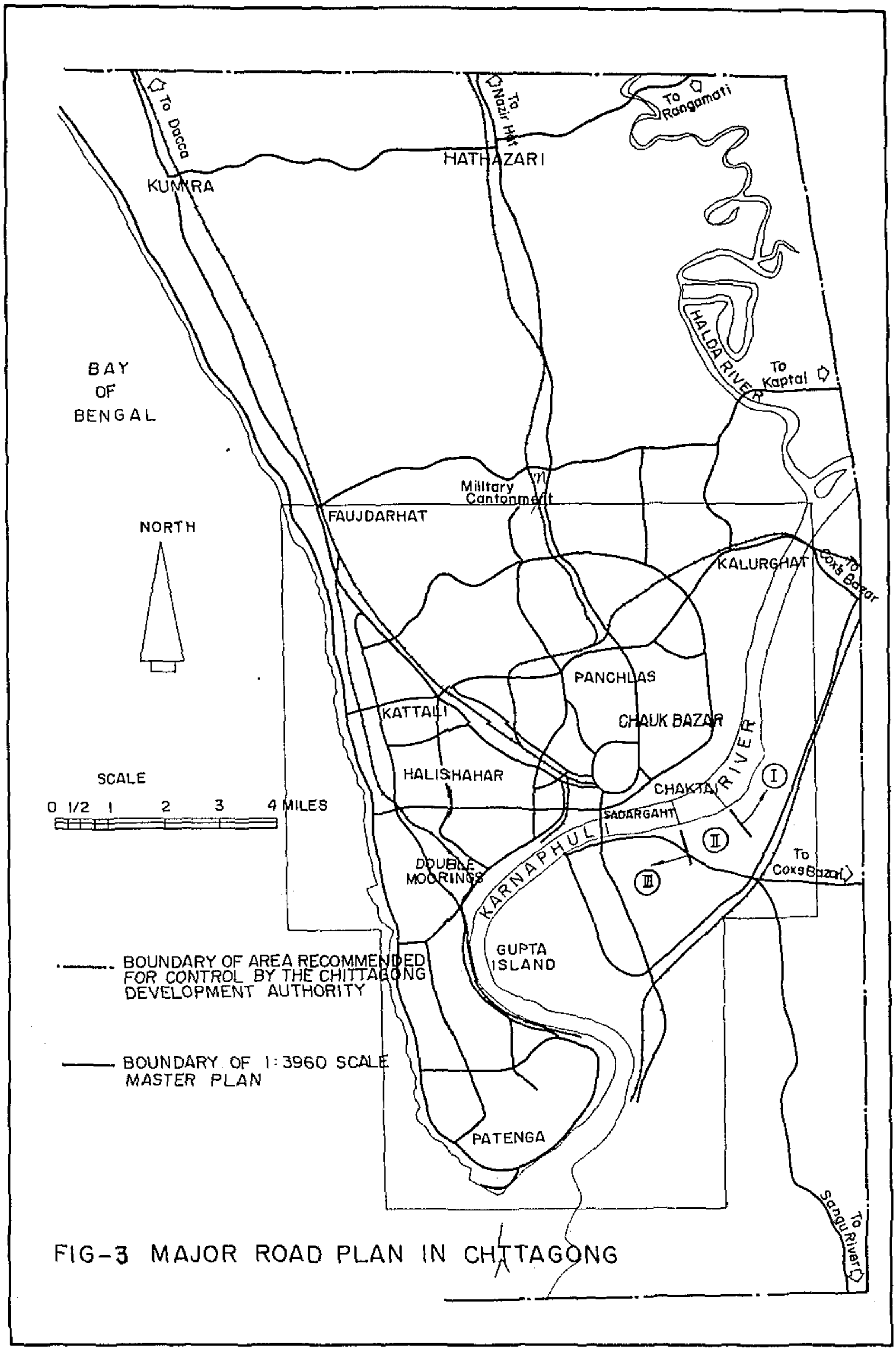


FIG-3 MAJOR ROAD PLAN IN CHITTAGONG

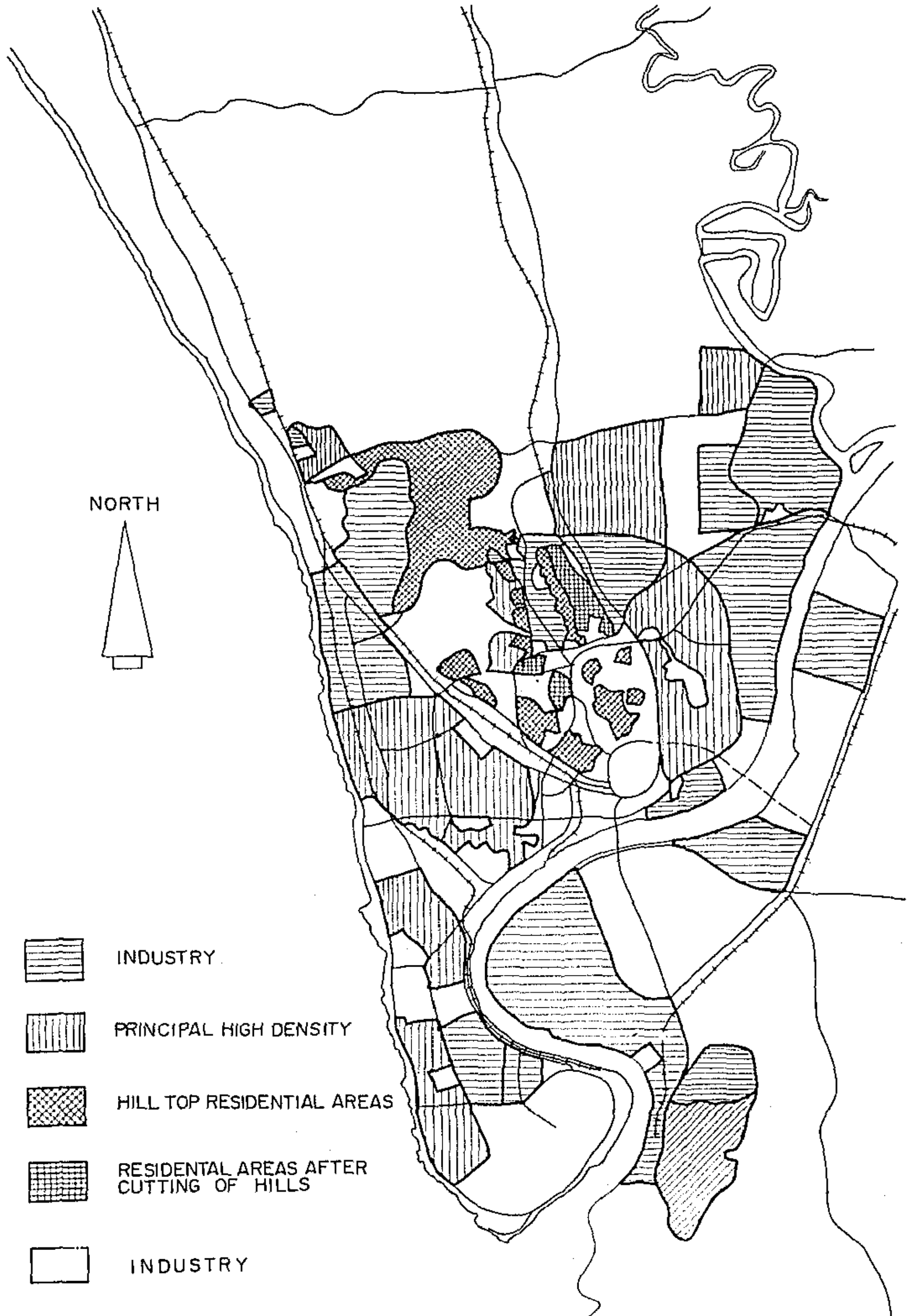


FIG - 4 MASTER PLAN IN CHITTAGONG

CHAPTER IV

SITE INVESTIGATION

1. Topographical Survey

1. 1. General

The proposed center line of bridge was decidedly set up on the ground, and the triangulation was conducted to connect the both banks of river. A traverse was set up, too, making use of its stakes to carry out the plane-table surveying for drawing up the plan. On the other hand, the leveling work, both profile and cross, was carried out to determine the land elevation.

The survey work was conducted by 3 Mission members, 1 assistant engineer, 1 surveyor and 5 survey khalasi.

The main survey instruments were as follows:

Transit Theodolite	2
Level	2
Plane Table	1
Measuring Tape (steel & vinyl)	2
Binocular	1
Transceiver	4
Current Meter	1
Echo Sounder	1

The surveying was begun on 29th January and finished on 23rd Feburuary.

1. 2. Setting-up of Proposed Center Line of Bridge

After several field investigation in addition to the study based on the map, a proposed center line of bridge was determined on map to make a right angle with the center line of the Karnaphuli, and it was set up in the field. The total 4 wooden stakes were posted on the proposed center line of bridge: The first (No. 74 + 34. 525) on the left bank of the small channel 1. 2 km from the right bank of the river, the second (△G) in the riverside of the right bank, the third (△ 0) in the shoal near the right bank and the fourth (△ E) in the riverside of the left bank.

1.3. Triangulation

As it was the most suitable from the topographical viewpoint to set the base-line in the shoal near the right bank, such triangulation net was made out as shown in Drawing I. 1. The stations of triangulation were installed: One station (△G) in the riverside of the right bank, four ones (△0, △ A, △ B, △ C) in the shoal near the right bank, one each (△ D, △ F) at the shallows in the river, one (△ E) in the riverside of the left bank, making 8 stations in total. For △ G, △ O, △ E, of these stations, the stakes on the center line of bridge were made use of. The stakes were wooden made, at the center on the top surface of which were nailed. The base line was set between △ O and △B, that was served as the check base-line at the same time.

Between △O and △ B there were 3 standard stakes driven at intervals of 46-48 m, and further driven were the total 4 supporting stakes at a regular interval between every standard stakes. With a steel tape of 50 m long, the measurements were made 5 times both ways from each point. At the same time, the measurements were made for the temperature and the elevation difference between every standard stake, so as to effect the correction for temperature and slope. The measurements were made at the tension of official approval (15 kg), and no correction, therefore, for tension was required. As the result given after all, the actual length of base line came out to be 189.704 m.

A transit of 20-second reading was used for the survey of angle by the reciprocal survey method repeating 3 times. The measurement was carried out clockwise in the normal and reverse states of telescope. The mean value was taken up as the measurement value.

A general approximation was used for the calculation of triangulation net. Each angle was adjusted by angle equations, and the angles resulted were corrected in order to satisfy side equations in which the actual length of the base-line was used. Six triangles of △OBC, △ OCD, △ ODE, △ OEF, △ OFA and △OAB were first adjusted to give the most probable value, and then the adjustment of △BAG and △BGC was worked out. Further, taken the north direction (magnetic north) as X-axis and the east direction as Y-axis, the co-ordinates of △O to be x = 2,000 m and y = 2,000 m, the co-ordinates of all the stations of triangulation were calculated. The results were as shown in Drawing I. 1. The proposed center line of bridge was given to be in the azimuth of N37°30''W.

1.4. Traversing

The survey field could afford an unobstructed view and the traversing was carried out only by extending the proposed center line of bridge along which the measuring was made by the steel tape at the interval of 40 m, the center stakes being driven at the place. The stakes were made of lumber and, the number of stakes newly driven was 54.

1.5. Leveling

About 700 m up the river from G point on the right bank, the standard bench mark was selected at G. T. C. B. M. 7 (Elevation: M. S. L. + 9.590 ft. = 2.923 m) on the route from B. M. 71 to B. M. 58 P. P. on the "Great Trigonometric Series" of Survey of Pakistan.

Four temporary bench marks were set up with concrete stakes as follows:

T. B. M. No. 1	Near △ E on the left bank
T. B. M. No. 2	Near △ O in the shoal near the right bank
T. B. M. No. 3	Near △ G in the riverside of the right bank
T. B. M. No. 4	Near the point No. 74 + 34.525 on the right bank

The leveling was conducted twice one-way. The elevation of the top surface of the center stake and the ground surface are shown in Table - 2.

Table 2 Elevation of Top of Center Stake and Ground-Surface (Above M. S. L.)

Left Bank

No. of Center Stakes	Distance (m)	Top of Stakes (m)	Ground Surface (m)
1		3.341	3.27
	14.0		
2		1.998	1.91
	15.0		
3		1.878	1.70
	11.5		
4		3.417	3.13
	40.0		
5		2.159	2.04
	40.0		
6		2.325	2.14
	40.0		
7		2.270	2.05
	44.0		
8		3.035	2.73
	36.0		
9		2.095	1.92
	40.0		
10		1.908	1.76
	40.0		
11		1.980	1.83
	40.0		
12		2.136	1.98
	40.0		
13		2.063	1.96
	39.0		
14		3.679	3.58
	41.0		
15		2.131	1.93
	40.0		
16		2.191	1.96
	40.0		
17		2.040	1.96
	40.0		
18		2.471	2.30
	40.0		
19		2.593	2.34
	40.0		
20 (E)		2.986	2.94
(T.B.M. No. 1)		(3.293)	

Shoal near Right Bank

No. of Center Stakes	Distance (m)	Top of Stakes (m)	Ground Surface (m)
46		1.766	1.62
	40.0		
47		1.450	1.30
	40.0		
48		1.670	1.52
(T.B.M. No. 2)	40.0	(1.875)	
49		1.893	1.74
	24.261		
0		1.849	1.73
	15.739		
50		1.831	1.68
	40.0		
51		1.802	1.52
	40.0		
52		1.902	1.70
	40.0		
53		1.910	1.66
	40.0		
54		1.897	1.42
	40.0		
55		1.875	1.62
	40.0		
56		1.897	1.60
	40.0		
57		1.656	1.40
	40.0		
58		1.875	1.68
	40.0		
59		1.858	1.71
	40.0		
60		1.978	1.83
	26.496		
60+26.496		1.934	1.80

Right Bank

No. of Center Stakes	Distance (m)	Top of Stakes (m)	Ground Surface (m)
62		2.245	2.08
	4.085		
G		2.223	2.16
(T.B.M. No. 3)	35.915	(2.336)	
63		2.146	2.07
	40.0		
64		2.457	2.30
	40.0		
65		2.572	2.40
	40.0		
66		2.614	2.46
	40.0		
67		2.600	2.45
	40.0		
68		2.537	2.38
	40.0		
69		2.326	2.17
	40.0		
70		2.646	2.43
	40.0		
71		2.589	2.41
	40.0		
72		2.541	2.39
	40.0		
73		2.568	2.37
	40.0		
74		2.654	2.50
(T.B.M. No. 4)	34.525	(3.537)	
74+34.525		3.078	2.98
	35.440		
75+29.965		3.109	2.96
	10.035		
76		2.295	2.10
	40.0		
77		2.343	2.32
	40.0		
78		2.587	2.44
	40.0		
79		2.609	2.41

Since the distance between the temporary bench marks on both banks of the Karnaphuli were measured about 4,000 feet, the leveling was made 5 times on each bank by the reciprocal leveling method with targets.

The longitudinal section of the ground-surface along the proposed center line of bridge, and the cross sections at the center stakes were surveyed. The results from these surveys were shown in Drawing I-2. Judging from the results of the leveling, the shoal stands lower about 0.8m than the right bank, and the left bank lower about 0.4 m than the right bank.

1.6. Plane-Table Survey

The plane-table surveying was made along the proposed center line of bridge for 110 - 150 m in width covering both upper and lower sides of the center line and in length for about 700m inside the left bank, about 600m inside the shoal near the right bank and about 700m in the right bank. The points for the plane-table setting were 10 on the left bank, 4 in the shoal and 5 on the right bank, totaling 19 setting points. As the result was given the plan at the scale of 1/1000, which is shown in Drawing I. 3.

2. River Survey

2.1. General

The Karnaphuli rises from the Assam District of India and flows into the Bay of Bengal at Patanga. The area of river basin is 5,500 squar miles, the length reaches 132 miles and the river width is about 3,000 feet at Chittagong.

The water surface at the time of full tide comes up to the top of river side. According to the C.P.T.'s survey that has been continued since 1922, the rivercourse has changed to a considerable extent. In 1962, the Kaptai Dam was completed about 53 miles upstreams from the river mouth, and, since then, the change in the rivercourse has been different from that in the preceeding time. At present, the rivercourse is to be controlled with the training works by the C.P.T.

The bridge construction site is situated directly downstreams from the curving point of the river. The deepest flow runs just in the center of river at present, but it is presumedly moving year to year.

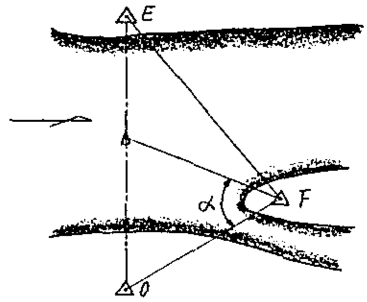
Among the field surveys were the sounding, measurement of current velocity, and observation of water level of river. Based on the findings of the survey and the existing data, the characteristic features of the river would be presumed to estimate the degree of scouring for the bridge foundation.

2 Survey Mission members engaged in the river survey, being accompanied by 1 surveyor and 4 assistant workers sent by the C.D.A.

2.2. Sounding

The sounding of the river had been continued by the C.P.T. for the period of 44 years from 1922 up to date, covering the distance from the confluence with the Halda River to the Chittagong Port, including the proposed bridge site. As the past records were available, the sounding of this time had only to be made along the proposed center line of bridge and the parallel lines 80m apart from it for both upper and lower sides.

The method of the sounding was as follows: The echo sounder was brought aboard a country boat. By means of sighting the survey flags, 2 each on both banks, the boat was led onto the proposed center line of bridge. On the other hand, a transit was fixed at the station △ F of triangulation as shown in the following picture, taking the angles ∠α that had been previously calculated at intervals of 20m on the proposed center line of bridge. When the sounder came to accord with the line of sight through the transit, the signal was sent by handly-talkies and the sounding was effected.

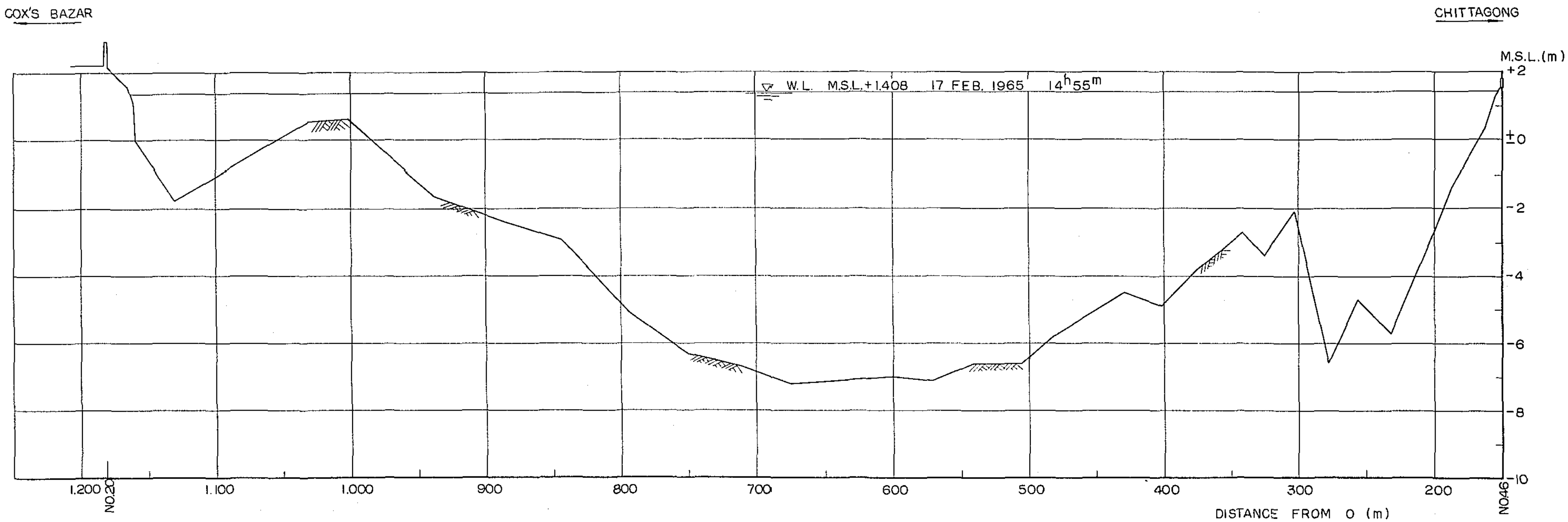


The influence of the tidal current upon the water level was measured by the water-gauge posted on the right bank, to correct the measured water depth. The deepest part lay about the center of river where was measured to be 7.00m.

2.3. .leasurement of Current Velocity

The current velocity was measured in the same way as in for the sounding, taking a country boat loaded with the echo-sounder and electric current meter to the measuring points by means of making use of the stations of triangulation. The boat was stopped at the point by its anchors down in the water. The measurement was made at each depth of 1 m at intervals of 40m across the river. It took time for the velocity measurement to be conducted, and eventually the water level should have to be modified by the reading of the water gauge. The velocity was measured at the time of the receding and coming tide respectively, resulting that the maximum velocity was 1.20 m/s at the receding tide and 0.98 m/s at the coming tide.

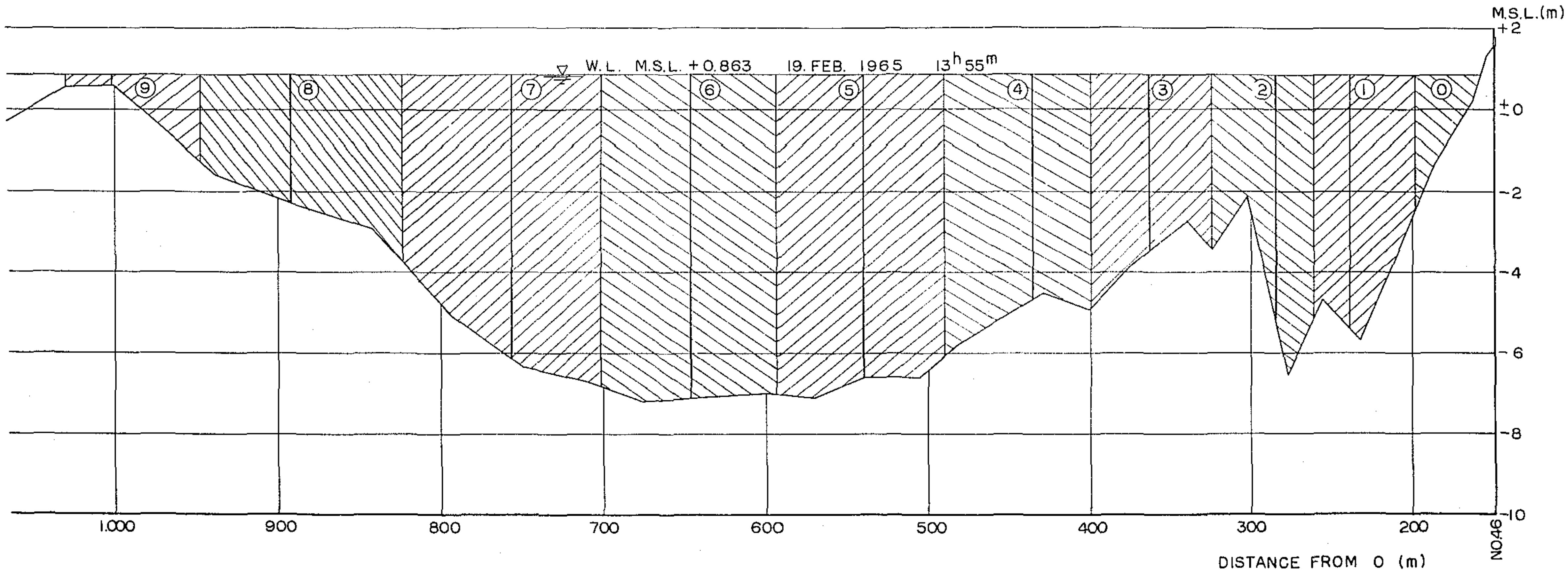
PROFILE OF KARNAPHULI RIVER AT THE BRIDGE SITE



RIVER SURVEY

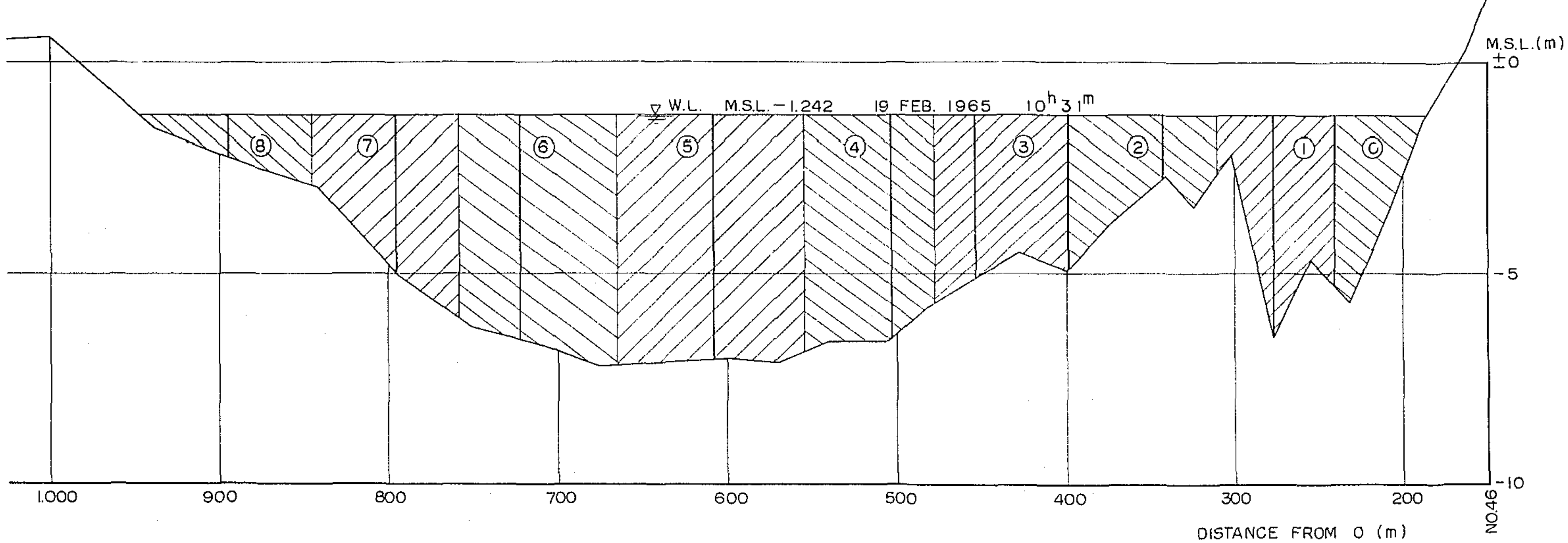
(FLOOD TIME)

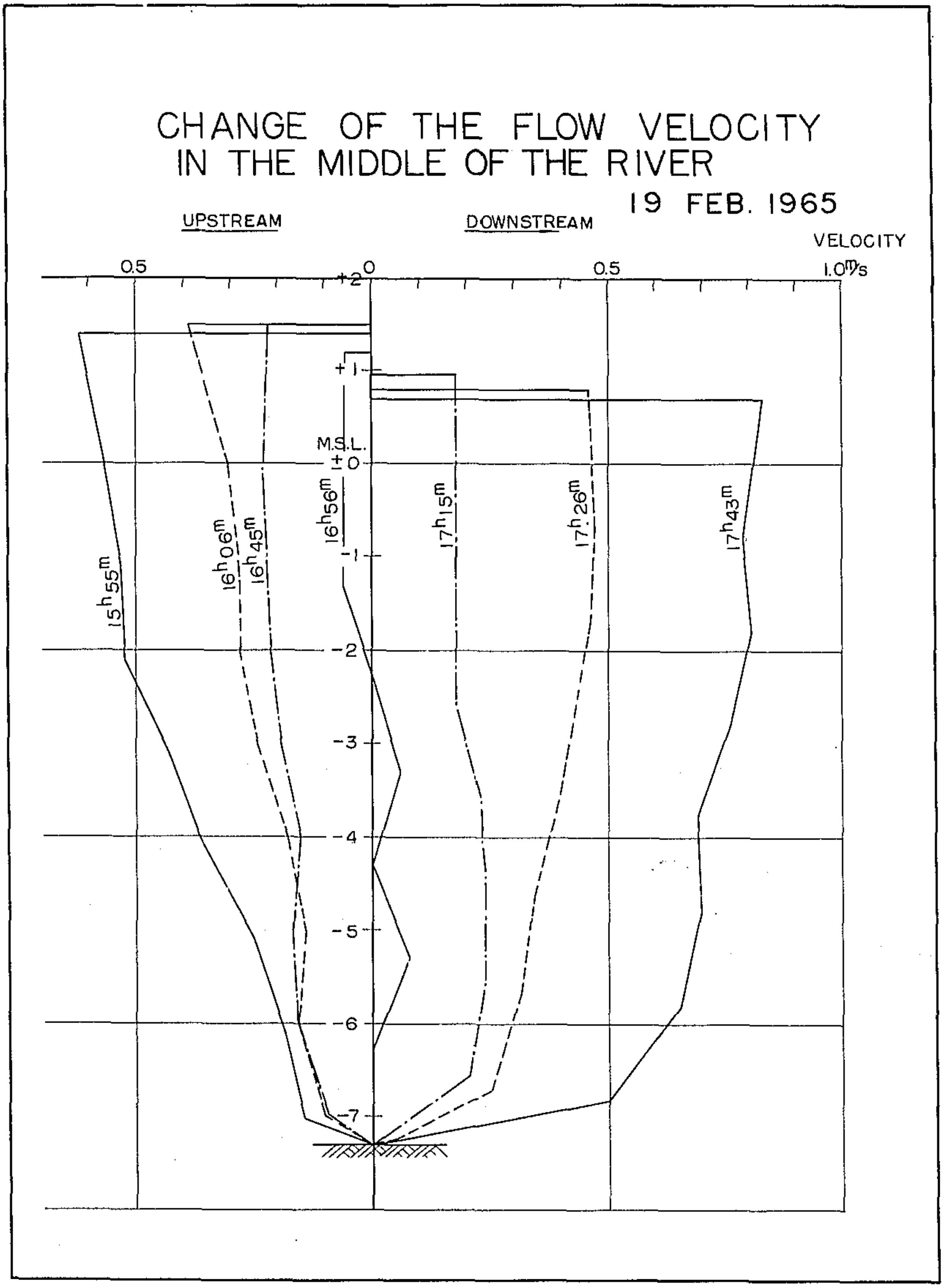
	VELOCITY	AREA	DISCHARGE
⓪	0 m/s	60 m²	0 m³/s
①	0.72	401	287
②	0.65	388	253
③	0.74	327	242
④	0.97	489	475
⑤	0.77	787	607
⑥	0.77	860	660
⑦	0.51	860	437
⑧	0.44	384	168
⑨	0.22	61	13
TOTAL		4.617	3.142



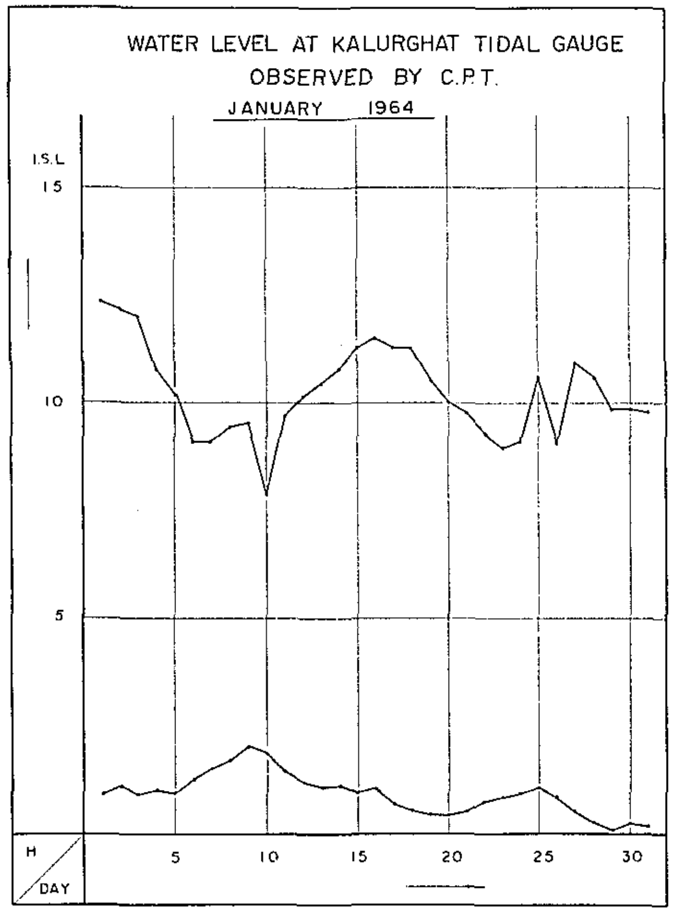
RIVER SURVEY
(EBB TIME)

	VELOCITY	AREA	DISCHARGE
⓪	0 m/s	108 m^2	0 m^3/s
①	0.32	138	44
②	0.69	128	88
③	0.77	309	238
④	0.85	406	348
⑤	1.06	634	671
⑥	0.81	499	404
⑦	0.55	325	179
⑧	0.34	147	50
TOTAL		2.694	2.019





CHANGE OF THE FLOW VELOCITY
IN THE MIDDLE OF THE RIVER
19 FEB. 1965
UPSTREAM
DOWNSTREAM
VELOCITY
0.5
0
0.5
1.0m/s
+2
+1
M.S.L.
±0
-1
-2
-3
-4
-5
-6
-7
15h55m
16h06m
16h45m
16h56m
17h15m
17h26m
17h43m


WATER LEVEL AT KALURGHAT TIDAL GAUGE
OBSERVED BY C.P.T.
JANUARY 1964
I.S.L.
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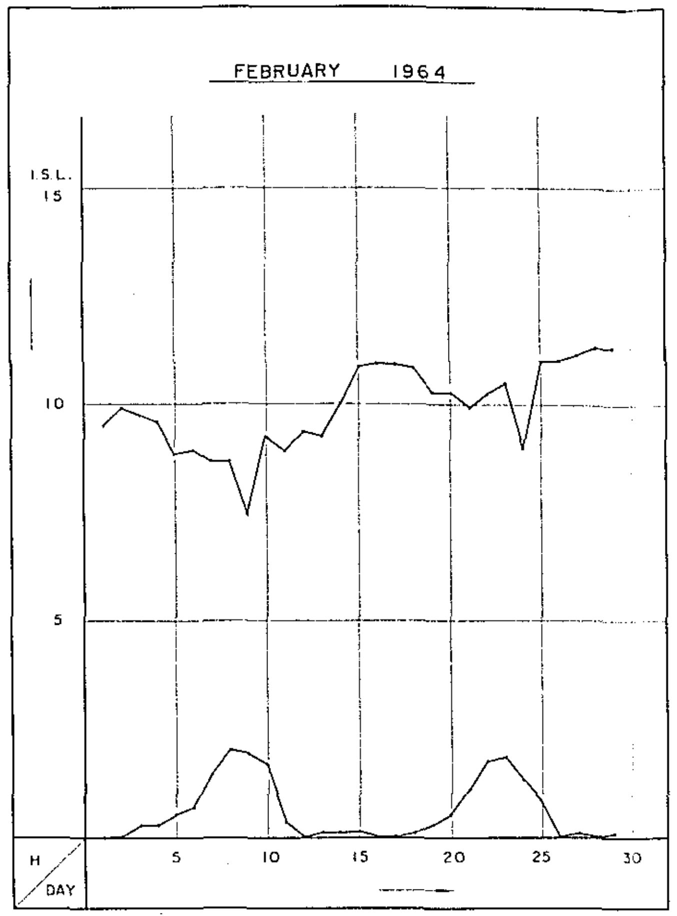
FEBRUARY 1964
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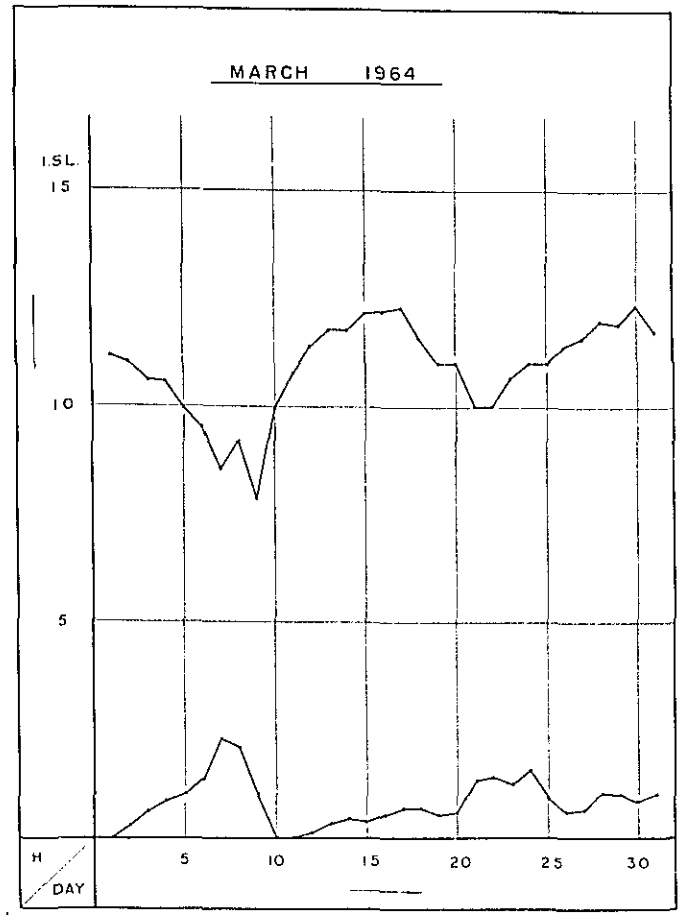
MARCH 1964
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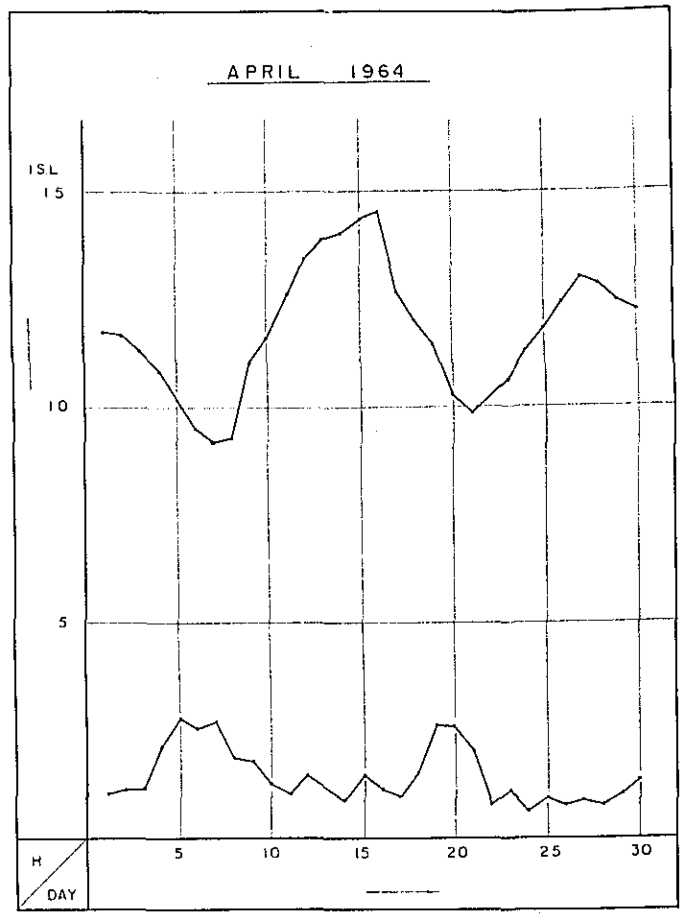
APRIL 1964
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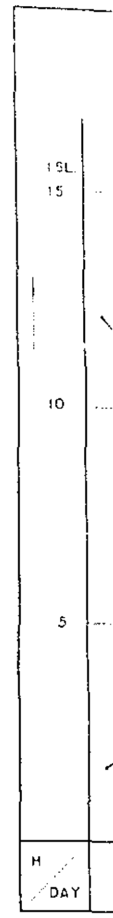

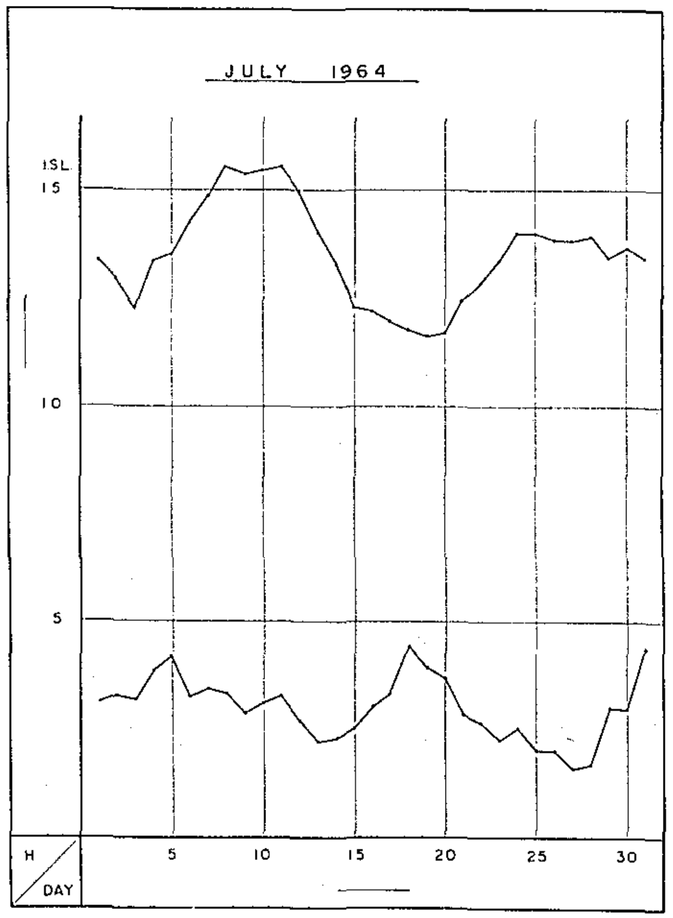
JULY 1964
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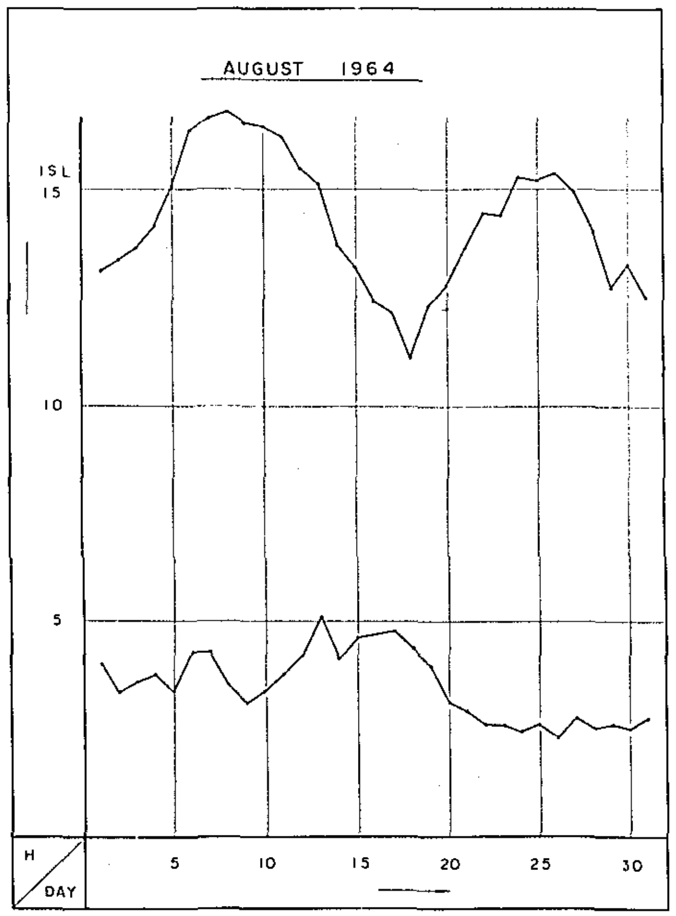
AUGUST 1964
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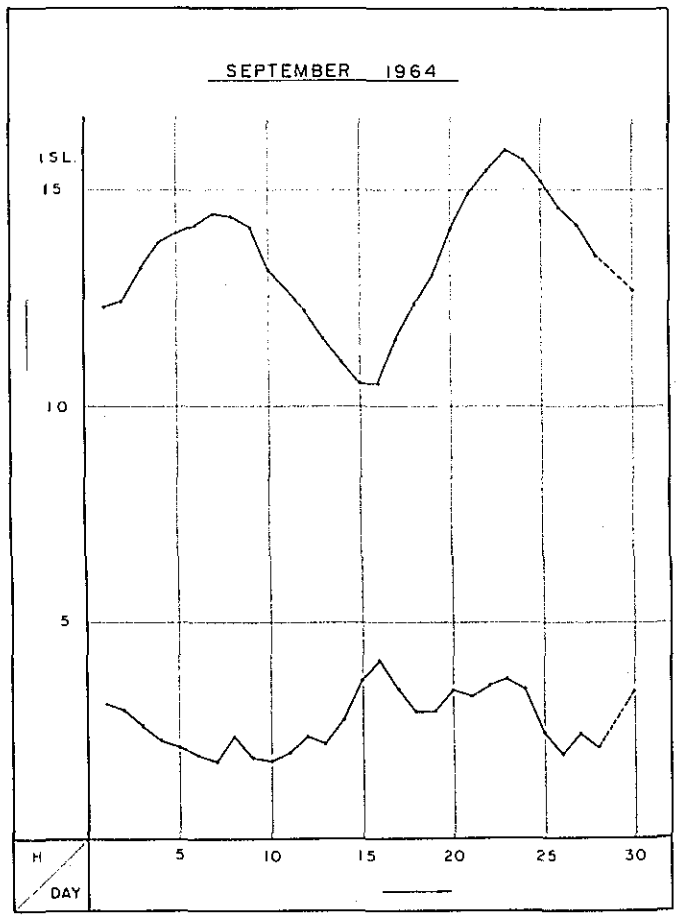
SEPTEMBER 1964
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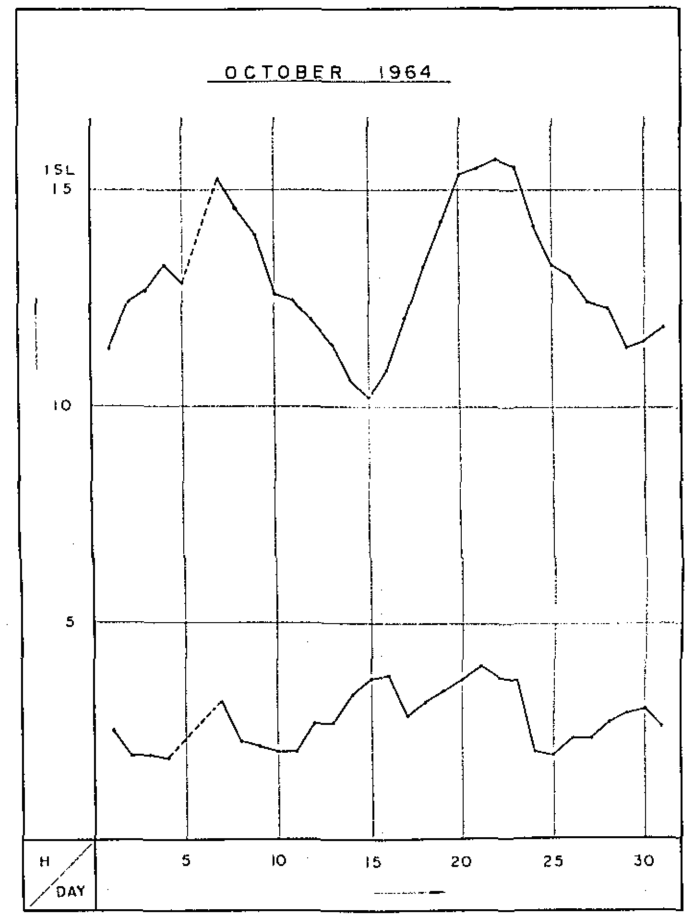
OCTOBER 1964
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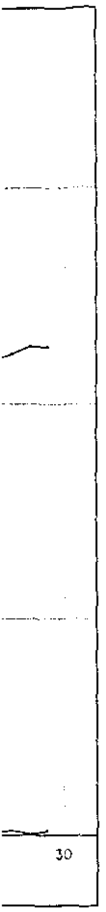
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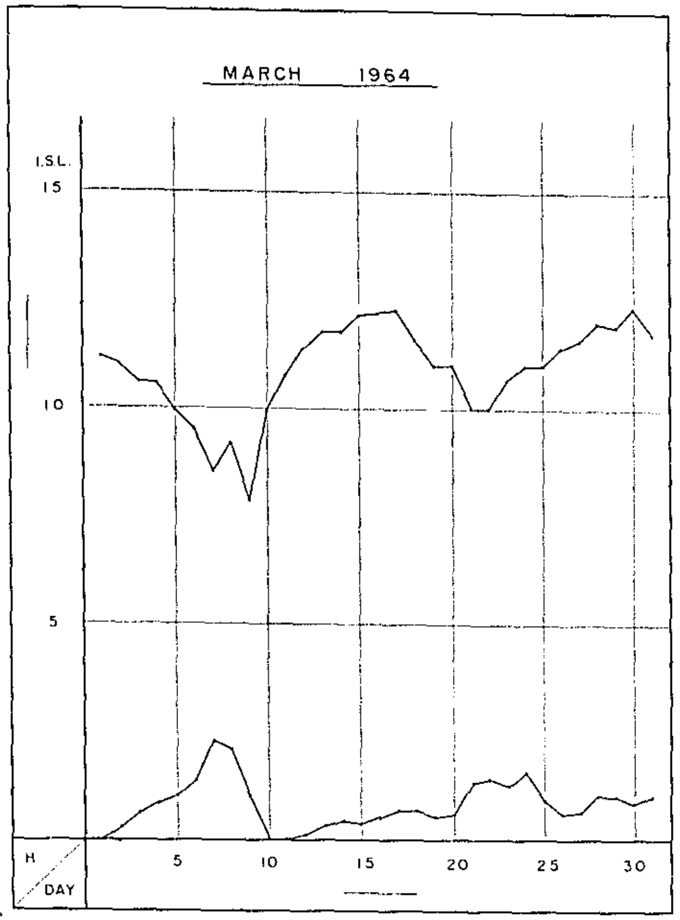
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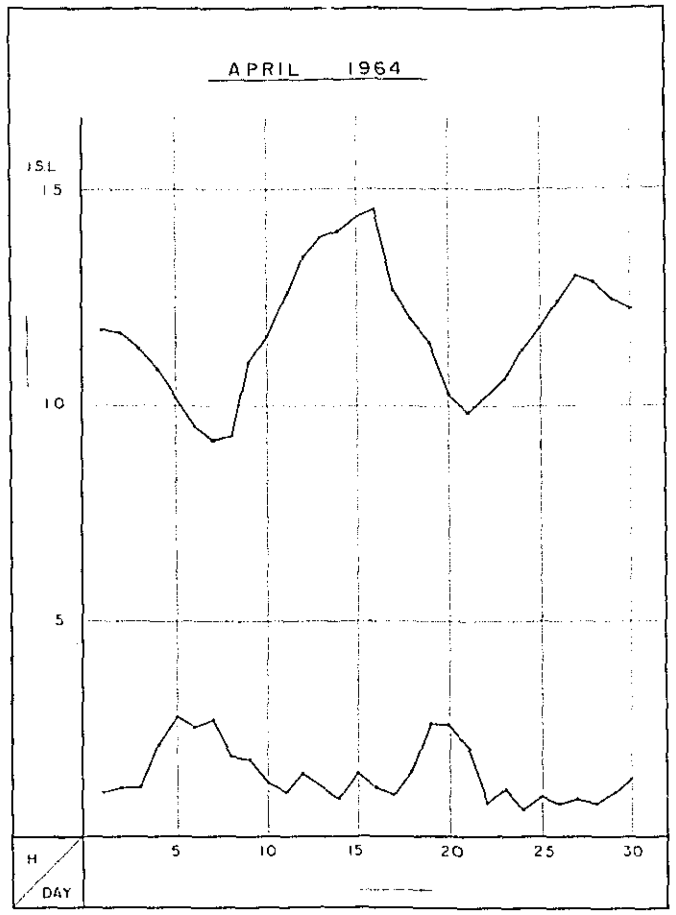
APRIL 1964
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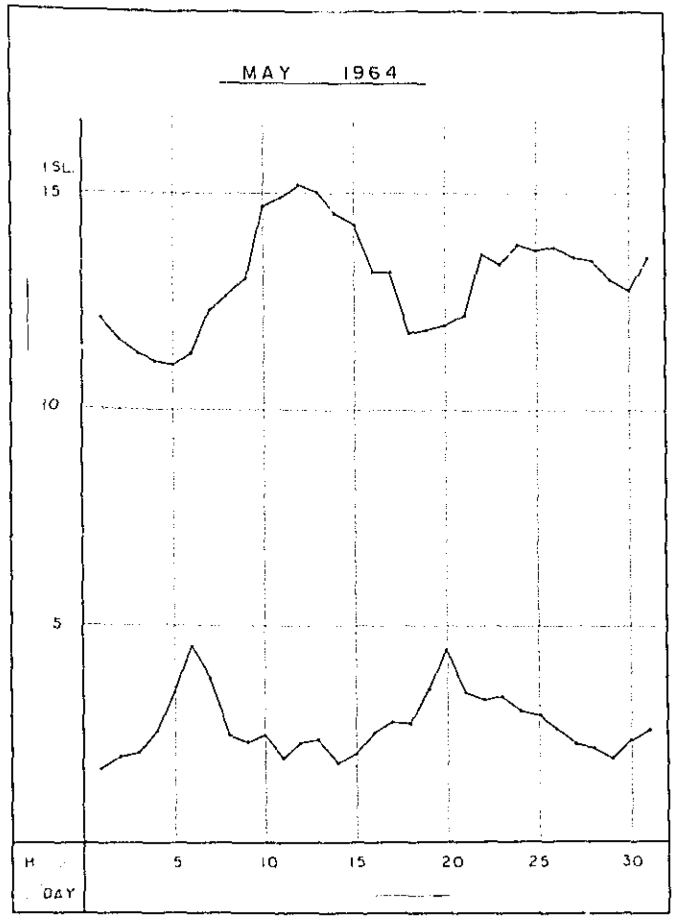
MAY 1964
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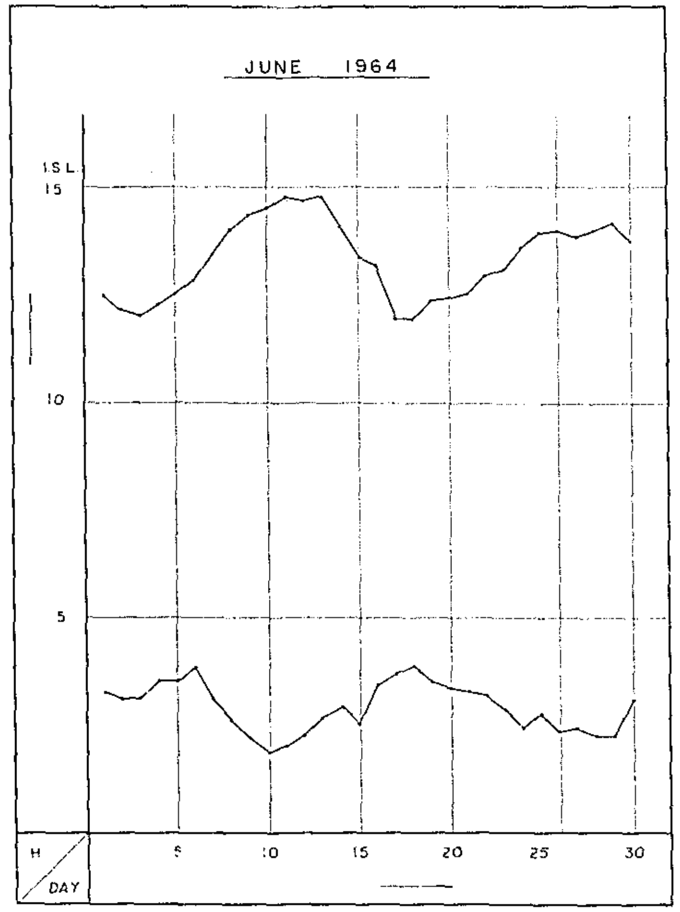
JUNE 1964
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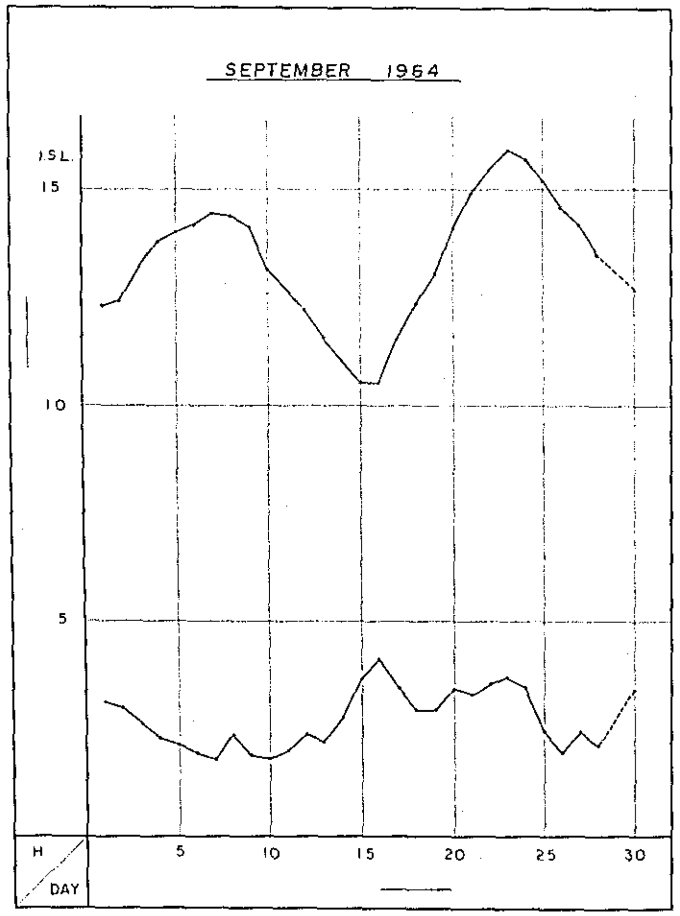
SEPTEMBER 1964
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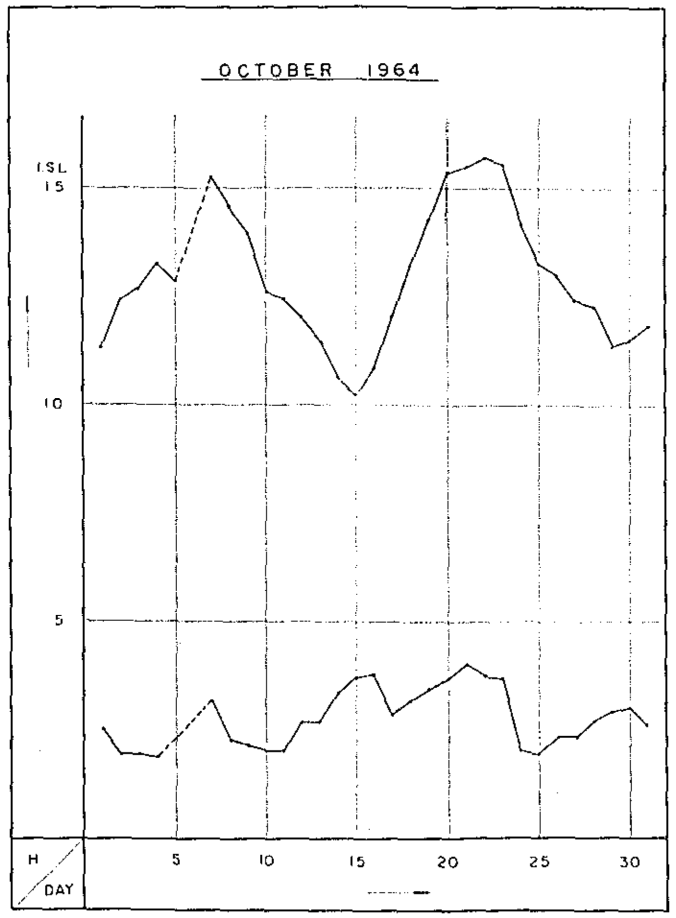
OCTOBER 1964
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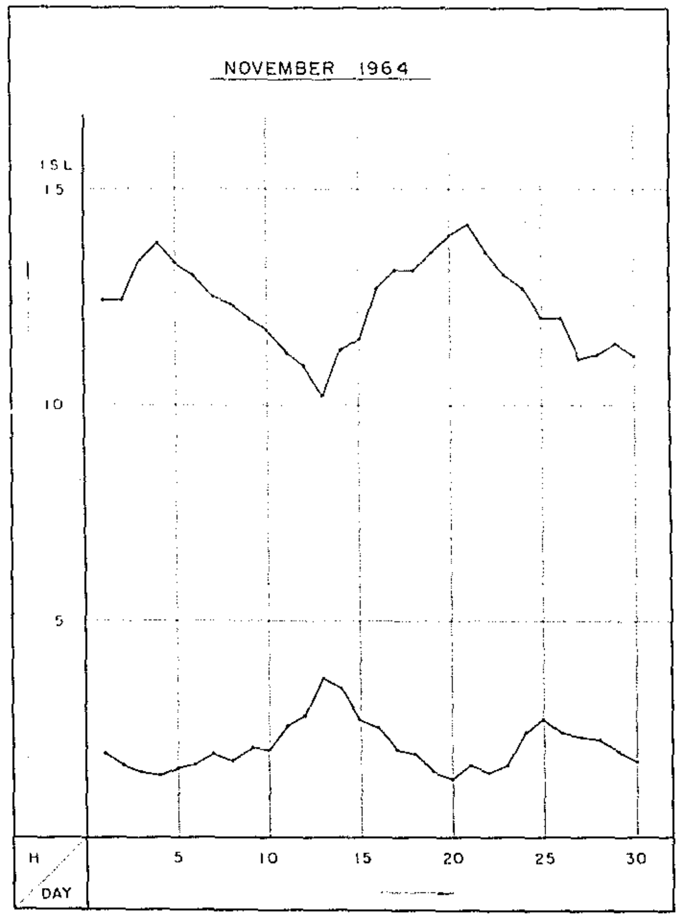
NOVEMBER 1964
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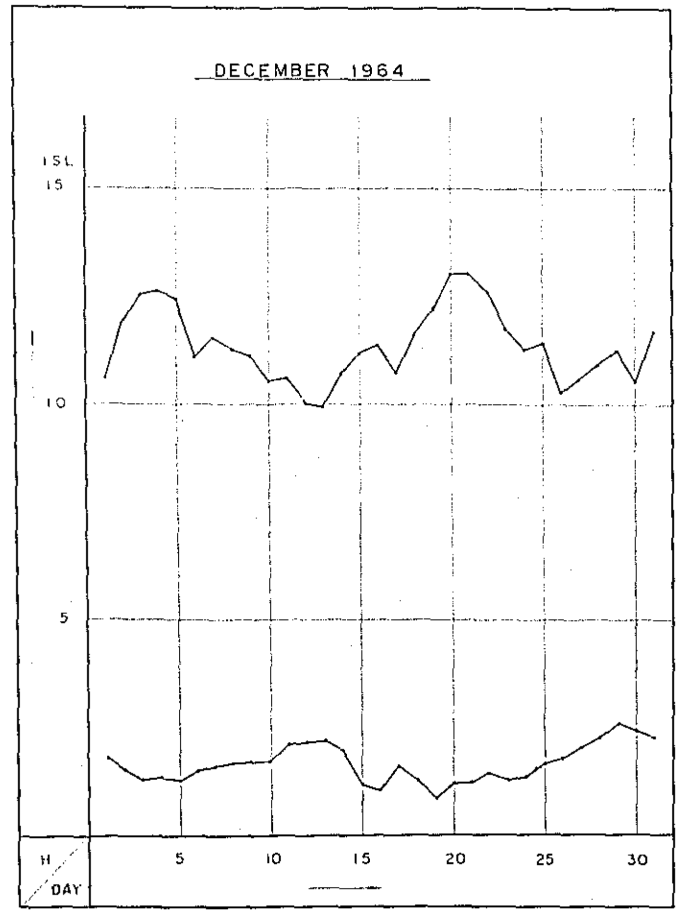
DECEMBER 1964
I.S.L.
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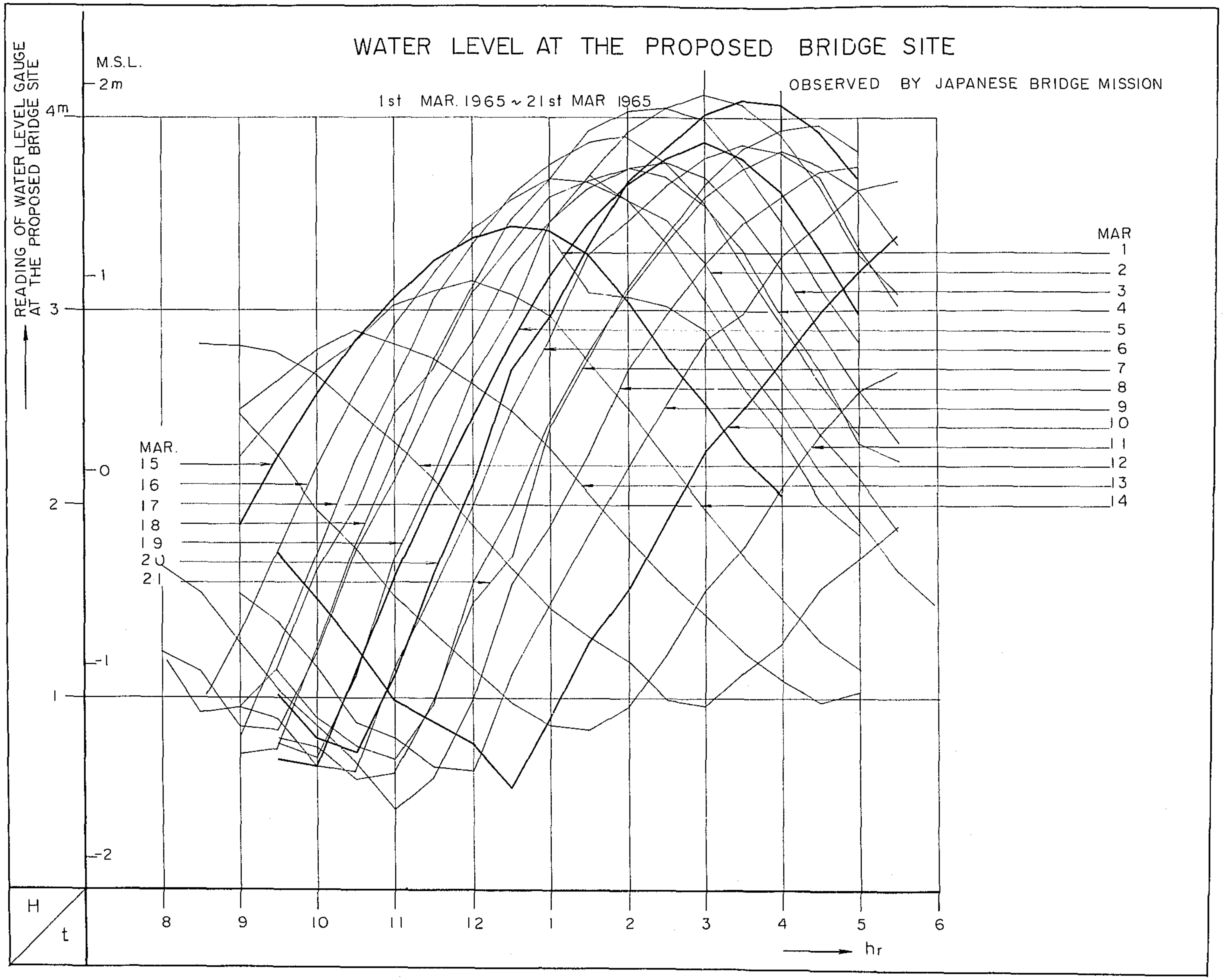
WATER LEVEL AT THE PROPOSED BRIDGE SITE
OBSERVED BY JAPANESE BRIDGE MISSION
1st MAR. 1965 ~ 21st MAR 1965
READING OF WATER LEVEL GAUGE AT THE PROPOSED BRIDGE SITE
M.S.L.
2m
1
0
-1
-2
4m
3
2
1
MAR
1
2
3
4
5
6
7
8
9
10
11
12
13
14
MAR.
15
16
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18
19
20
21
H
t
8
9
10
11
12
1
2
3
4
5
6
hr

3. Subsurface Exploration

3. 1. General

The subsurface exploration was made by means of the standard penetration test and vane test in the site, and the laboratory test with the soil sample. The in-situ test and the sampling were carried out at the required depth with the aid of the boring. The number of boring point were 9, and the total boring depth was 500 m.

The working period of the field test began on 28th January and ended 28th March, taking 60 days in all. The soil test in Japan was carried out for the period from 10th April to 31st May.

The Pakistani staff members jointed the Survey Mission in the boring work: the executive engineer and assistant engineer of the C. D. A. at times, and 2 surveyors of the C. D. A. throughout the working period. In addition to the abovementioned members, 1 assistant worker, 6 laborer and 4 night watchmen were employed.

3. 2. Outline of Geological Feature

At the estuary of the Karnaphuli, there is a plateau holding the river mouth, and the movement of the river mouth has been hindered causing little growth of a typical delta plain. In the upper streams from the plateau, however, the considerable changes in the water-course have been taken place. Chittagong City stands on the diluvial plateau to the right bank of the river. The bridge location situated out of the outstretch of the diluvial plateau, and the left and right bank areas at the site are in the alluvial plain, most parts of which are now cultivated for rice field.

The land on the right bank has been lately formed and that may as well be called a shoal. Besides, the river represents an extreme meandering flow, and its main water-course has been more often changed. The width of river is 3, 000 feet and the main watercourse runs in the central part of the river, but it seems not to be stabled. The trace of the past watercourse is found about 3, 000 feet inside from the present stream, remaining as a small channel. In the lower stream, the width of the river becomes narrow down to 1, 500 - 2, 000 feet, owing to the diluvial plateau protruding to the river, so the upper stream area is likely to be easily flooded.

3.3. Decision of Exploration Plan

The points of subsurface exploration were determined from such viewpoints that the overall geological characteristics of the foundation should be made clear rather than the study of the soil nature only at the points of bridge piers, chiefly because the bridge composition had been unknown yet.

The first thing was to drill two bores on each bank for the depth of 50 - 60 m. The boring work resulted in showing: - The surface layer of 2 - 3 m thickness consists of clay and silt containing organic materials, below which the alternation of medium and fine sand was found to be dominent, sometimes the clayey-soil thin layer's being held between them. The gravel was seen about 2 m thick at the depth of -50 m. Concerning the bearing strength of the ground, the N-value obtained from the standard penetration test was 30 at the depth of -10 m, 40 at -20 m, and over 70 at -40 m. Sometimes when encountered the silt stratum, it would come down to 30 or less. It was proved that these geological characteristics made little differences between the left and right banks, and it was presumed that the similar features should exist for all the area of the proposed survey field.

Consequently, the subsurface exploration plan as follows: -

For the portion of the bridge construction site, 3 bores should be drilled in the river (at intervals of 250 m and for the depth of 70 - 100 m) in addition to 2 bores on land on both banks. Especially, the boring at the middle point in the river should be made so as to reach the diluvial foundation, the boring depth being taken 100 m. For the portions of the approach road which would have an extensive length, the short borings were planned to be made at 4 points.

3.4. Method of Exploration

For the boring work, the TONE MODEL UD-5 boring machine was used, the bentonite milk being sent throught the cross bit of 86 mm in diameter in order to protect the wall of bore hole and wash away the slime. One of the difficulties met in the boring work was that in the river it should have to be done under such unfavourable conditions as a large tidal range (3 - 4 m) and fast current flow (about 15 m/s at the maximum). The work was after all done successfully, using 2 country boats of 25-ton capacity fastened to each other and fixing them by 4 anchors of 120 kg at the point in the river.

The standard penetration test was undertaken with the Reymond sampler (total length 810 mm, outside diameter 51 mm, inside diameter 35 mm), which was fixed at the end of the rod and brought down to the bore bottom. A certain blow was applied on the top of the sampler (free falling of a drop hammer of 63.5 kg in weight from the height of 75 cm). The count was taken for the number of blow (N) required to let the sampler into the specified depth (30 cm). With the N-value, the relative density and relative consistency of the soil at the original place could be obtained. The test was conducted at intervals of 1-3 m all the way down to the depth of 100 m, and at the same time the disturbed soil sample was collected for the soil test.

The undisturbed soil sampling was carried out with the thin wall sampler (inside diameter 72.5 mm, outside diameter 75 mm, length 800 mm) and Dennyson sampler. The vane test was conducted to obtain the shearing strength and sensitivity ratio of the soft clayey soil. The portable vane-shearing tester for field work was used (strain-control, gear-winding type, with maximum torque 750 kg/cm; cross-wing of height 100 mm, width 50 mm, thickness 1.5 mm). The cross-wing was pushed down to the bottom of bore, where being rotated at the rate of 10 degrees per minute, and the torque was recorded. Then, it was rotated 20 rounds enough disturbing the soil along the shearing plane, and the same operation as previously done was repeated to obtain the sensitivity ratio in the soft clayey soil.

The soils sampled in the boring work were put to the following tests in accordance with the JIS (Japanese Industrial Standards) specification.

Moisture content test

Specific gravity test

Mechanical analysis

Consistency test

Bulk density test

Unconfined compression test

Triaxial compression test or Direct shear test

Consolidation test

Among the tests that were conducted in Pakistan were the moisture content test, specific gravity test, mechanical analysis, bulk density test, unconfined compression test. The rest was tested in Japan with the undisturbed sample, which was sealed by paraffin in an anti-shock package and sent by air.

3.5. Findings

Table 3 Boring Points

Boring No.	Distance (m)	Elevation (M.S.L.)(m)	Boring Depth (m)	Status in Dry Season	Remarks
B1	0	+1.93	40	Farm	Left Bank (Cox's Bazar side)
B2	250	+1.67	50	Riverside	Submerged in water in full tide.
B3	245	-1.40	70	In river	Max. depth 5 m at survey time.
B4	250	-7.00	100	"	Max. depth 11 m at survey time.
B5	242	-5.10	70	"	Max. depth 9 m at survey time.
B6	302	+1.62	60	Riverside	Submerged in water in full tide.
B7	302	+1.66	40	Paddy-field	Shoal
B8	246	+1.72	40	"	"
B9	200	+2.35	40	Farm	Right Bank (Chittagong City side)

The geological characteristics, sequence and thickness of strata were found as shown in the following Table-4. The soil profile is shown in Drawing I. 4.

Table 4 Sequence and Thickness of Strata

(meter)

Boring No.	B1	B2	B3	B4	B5	B6	B7	B8	B9
1. Clayey soil containing organic materials	2.50	4.60				5.40	3.90	4.80	4.30
2. Upper fine & medium sand	15.30	13.10	14.60	10.80	13.50	25.70	22.90	19.20	21.60
3. Upper silt & fine sand alternation	5.90	6.90	9.90	11.20	10.50	7.30	5.00	4.50	over 5.80
4. Lower fine & medium sand	7.80	13.40	8.50	9.60	11.00	4.20	over 8.60	9.50	
5. Lower silt & fine sand alternation	4.50	4.40	5.50	3.70	4.50	1.70		over 2.30	
6. Upper medium & coarse sand	over 4.75	3.60	4.30	6.90	5.00	3.70			
7. Gravel		1.50	2.20	3.80	2.00	1.40			
8. Lower medium & coarse sand		over 3.30	over 26.60	over 55.56	over 23.93	over 11.90			

(1) Stratum of clayey soil containing organic materials

Distributed for the depth of 2.5 - 3.0 m in the surface layer but not in the river. Consists of clay and silt clay containing a quantity of organic materials such as wooden chips and grass roots. Dark grey or dark bluish grey in color.

The moisture content is 40%. The consistency ranges widely from "very soft" to "stiff".

Table 5 Characteristics of Clayey Soil

Boring No.	B1	B2	B3	B4	B5	B6	B7	B8	B9
Moisture Content (%)	42.1	38.0	-	-	-	44.6	43.0	40.2	46.8
Bulk Density (g/cm^3)	1.83	1.84	-	-	-	1.74	1.85	1.83	1.74
Unconfined Compression Strength (kg/cm^2)	1.40	1.28	-	-	-	0.60	1.18	1.33	1.42
Cohesion (kg/cm^2)	0.11	0.14	-	-	-	0.12	0.15	* 0.28	0.37
Internal Friction Angle (deg. min.)	2-11	2-32	-	-	-	3-37	2-30	* 4-00	2-31
Void Ratio	1.07	1.06	-	-	-	1.23	1.01	1.09	1.27
Preconsolidation Load (kg/cm^2)	0.15	0.50	-	-	-	0.50	0.40	0.41	0.53
Compression Index	0.173	0.250	-	-	-	0.378	0.305	0.259	0.295
Coefficient of permeability (10^{-6} cm/sec)	2.50	8.25	-	-	-	0.57	3.29	7.90	0.65

* By direct shear test.

(2) Upper fine and medium sand stratum

Distributed in the entire survey field, partially containing organic material and thin layers of clayey soil. In bluish grey color. The thickness of stratum is 15 - 20 m. The natural moisture content is 30% on the average. The upper part is found slightly loose and the lower part rather tight. The mean N-value is 28, and the relative density is "medium".

(3) Upper alternation of silt and fine sand

Distributed in the entire survey field, partially containing organic materials. Fine sand in bluish grey and silt in bluish-green grey color. The thickness of the stratum is about 5 m at B1 and B9, and more than 10 m at B4 and B5 in the central part of the river. The thickness of silt layer and sand layer ranges from 5 cm through 50 cm. The N-value is 27 for silt, and over 50 for sand. The result of soil test of the silt is given as follows: -

Table 6 Characteristics of Upper Alternation of Silt and Fine-sand

Boring No.	B1	B2	B3	B4	B5	B6	B7	B8	B9
Moisture Content (%)	34.7	35.2	41.3	46.8	39.5	40.6	32.4	36.5	39.8
Bulk Density (g/cm^3)	-	1.85	1.74	1.92	1.88	-	-	-	-
Unconfined Compression Strength (kg/cm^2)	-	1.68	3.00	2.78	2.63	-	-	-	-
Cohesion (kg/cm^2)	-	* 0.25	* 0.40	0.14	* 0.33	-	-	-	-
Internal Friction Angle (deg. min.)	-	* 14-31	* 8-05	7-11	* 9-29	-	-	-	-
Preconsolidation Load (kg/cm^2)	-	1.72	1.42	-	1.65	-	-	-	-
Compression Index	-	0.228	0.295	-	0.228	-	-	-	-
Coefficient of Permeability (10^{-6}cm/sec)	-	0.30	0.17	-	0.28	-	-	-	-

* By direct shear test.

(4) Lower fine and midium sand stratum.

Distributed in the entire survey field, and its thickness ranges from 5 to 15 m, partially containing organic materials and thin layer of silt. Generally the grain of sand is small in the part near the left bank, and larger near the right bank coming out in the medium sand. In a bluish grey color. Moisture content is 29% on the average, and often under 20% for medium sand. The N-value reaches as large as 58 and the relative density might be referred to as "very dense".

(5) Lower alternation of silt and fine sand

Distributed in almost entire field but not in B7 and B9, containing some organic materials at B4 and B5. In bluish grey color. The thickness of stratum is approximately 5 m. The average moisture content is 31%, being much lower than the aforementioned stratum (3) upper alternation of silt and fine sand. The N-value is more than 70 for fine sand layer and 60 for silt.

(6) Upper medium and coarse sand stratum

As it made no appearance at B7 through B9 within the drilling depth, its distribution could not be confirmed. The thickness of stratum was approximately 5 m, partially interposing silt and containing granule. The color is bluish grey. The N-value shows over 70.

(7) Gravel stratum

Accumulated at the depth of -40m - -50m at B2 - B6, making a stratum of 2 - 4m thick. The ratio of pebble is 20 - 40%. Most of pebble is angular-shaped and consists of the sandstone and shale which belong to the tertiary deposit developed in the Chittagong Hill Tracts. Round-shape pebble of chart is sometimes found. The grain size is 5 - 10 mm. Among the strata showing little change each other,, this is the only key bed.

(8) Lower medium and coarse sand stratum

Accumulated continuously below the stratum (7) gravel stratum down to - 110m at the deepest bottom of the drilling. Bluish grey to bluish green in color. It contains thin silt layers. Uncarbonated wood-chips were found in B4 at the depth of -70m and -90m. The N-value is over 70 as such in the case of the upper medium and coarse-sand stratum.

As the results of the survey and test, the geological features of the survey field was proved to be adequately suitable for such foundation of bridge, banking and other structures as now in plan.

Taking into account that the N-value of bridge foundation should stand about 60, it was considered that the following points should be taken up for the bearing strata.

B3	El. -30 m
B4	El. -29 m
B5	El. -30 m

Under the abovementioned depth, no stratum exists that should possibly cause a secondary consolidation settlement.

CHAPTER V

STUDY OF DESIGN CONDITIONS

1. Traffic Volume

The estimation of the expected traffic on the new bridge might be obtainable from the summing up of (1) traffic transferred from the railway bridge and ferry-boat, (2) traffic produced by the development of the left bank area, and (3) expected traffic in case of the Asian Highway A-2 Route's completion. On the assumption that the bridge will be completed in 1971, the traffic is estimated as of 1981 and 1991, respectively 10 years later and 20 years later from the completion of bridge.

(1) Traffic Transferred from the Kalurghat Railway Bridge.

The Kalurghat railway bridge was constructed in 1930. In 1963, the pavement was added setting floor-slabs at the both outsides and middle of rails, so that it could serve as a one-lane road bridge. However, there should be a certain limit imposed on the traffic because the bridge allows only the one-way traffic and it takes time to wait for the train passing. Furthermore, the traffic is suspended at night. The bridge is a toll one with the fare of Rs. 5. 0. for bus and truck, Rs. 2. 5 for small-size automobile and Rs. 1. 0 for autobicycle. Pedestrian passage is prohibited.

The present traffic on the bridge and the content of it are as follows:

Annual traffic	80, 323
Mean daily traffic	220
Maximum daily traffic	360 (June 6, 1964)
Passenger car	32. 3%
Bus	46. 8%
Truck	20. 9%

The increase rate of traffic is obtainable from the record of the passage fare collection which shows the rate of 37% from 1962 to 1963. The rate comes to accord with that of the registration number of vehicles in Chittagong City in the same year.

Therefore, the traffic on the Kalurghat Bridge will be estimated from the registration number of vehicles, which is on the rapid increase for these years fluctuating from year to year. Being classified by type, the light vehicles (inclusive of auto-rickshaw and motor cycle) take 60-70% of the total number, but they will be gradually replaced by large vehicles in future.

Based on the increasing rate covering all kinds of vehicle, the expected number is estimated: 2,940 in 1971, 5,880 in 1981, and approximately 9,700 in 1991. Accordingly, the expected traffic is given as follows: -

	Registered number	Annual traffic	Daily traffic
1964	1,715	80,323	220
1971	2,940	138,000	380
1981	5,880	270,000	740
1991	9,700	455,000	1,250

Since the origin-destination research has never been conducted for the Kalurghat traffic, the transferable traffic, following the completion of the Karnaphuli River Bridge, is not to be presumed. However, a safe presumption might be made that the traffic would probably transferred to the new bridge for 50% of the total volume. As the industries are making development, the railway transportation would necessarily increase, making impossible to allow the vehicle passage on the bridge. It is therefore assumed that all traffic would be transferred to the new bridge after 1981.

Table 7. Transferred Traffic from Kalurghat

1971	190	(vehicles per day)
1981	740	
1991	1,250	

(2) Traffic Transferred from Ferryboat.

The people's traffic between both banks of the river is new dependent on the so-called "country-boat" ferry, which was owned privately by an individual. According to the C. D. A's data that was investigated for the purpose of planning a pontoon bridge construction, it was 20,000 persons per day. These people are mostly inhabitants in the river side, and will probably continue to use the ferry service to a considerable extent after the bridge is constructed. On the other hand, keeping pace up with the development of industries, the conveniences by bus, truck and car will increase to lower the ferry use eventually. Therefore, 40% out of those now using the ferry will be supposedly transferred to the bridge traffic at the time of bridge completion; 50% in 10 years later and 60% in 20 years later.

The number of the ferryboat passenger is tentatively presumed that it should make a proportional increase to that of population in the east bank area. In the Master Plan, population is estimated for each district with the average increase rate of 2.25% per annum:

	Chittagong City & Suburbs	East Bank	Surrounding Districts	Total
1961	370,000	127,700	150,296	647,996
1971	460,000	162,000	190,000	812,000
1981	578,000	199,000	280,000	1,012,000
1991	720,000	250,000	280,000	1,250,000

	Ferry Population	Passenger on Bridge
1971	22,000 (per day)	8,800 (per day)
1981	26,000	13,000
1991	31,000	18,600

Of the passenger number, 80% would take bus, 10% drive passenger cars and other vehicles, and the rest 10% be on foot through the bridge; it is made for the purpose of the calculation. (in converting into the vehicle number, 20 persons/vehicle for bus and 4 persons/vehicle for car).

Table 8. Transferrey Traffic from Ferryboat

	Bus	Others	Total	
1971	350	220	570	(vehicles per day)
1981	520	325	845	
1991	750	465	1,215	

(3) Introduced Traffic Following the Industrial Zone Development

In the Master Plan, the factory area on the left bank is scheduled to expand on the site of 5,371 acre. Now it is presumed that a factory area will develop on the land of 2,000 acre by 1991. Among the planned industries are the heavy industry with iron-works as its main plant, light industries, and the processing industries for agricultural products. The dependence degree of the in-bound and out-bound cargoes for overland transportation is made assumedly to be 35%, of which 70% is also presumed to be sent through the new bridge. The calculation is made on an assumption: the cargo volume per unit area of factory site to be 7,350 tons/year/acre. Provided a truck is of 6-ton loading capacity, and that the loading ratio is made about 70%, the required number of carrying truck will be 2,400 vehicle/day. The factory construction plan is dividedly considered in every 10 years.

	Developed Industrial Zone	Generated Traffic Volume (in-and out-bound)
1971	400 (acre)	960 (daily)
1981	1200	2,900
1991	2000	4,800

Supposed that 30 persons work at the unit factory area of 1 acre and 30% out of the people come from the right bank area, it makes 18,000 heads in 1991. The people are classified by the transport facilities that they use, as follows: -

	1971	1981	1991
Bus	3, 120	9, 505	16, 020
Car	70	270	540
Light Vehicle	320	865	1, 260
On Foot	90	160	180
Total:	3, 600	10, 800	18, 000

The number of people will be converted into that of vehicle as follows: (Coming in and out calculated)

	1971	1981	1991
Bus (20 passengers)	312	952	1, 602
Car (1 person)	140	540	1, 080
Subtotal	452	1, 492	2, 682
Motor-cycle (1 person)	640	1, 730	2, 520

The cars for commercial use in direct connection with factory production is assumedly 20% of the number of truck.

	1971	1981	1991
Car	190	580	960

The total of the introduced traffic following the industrial zone development becomes as follows: -

Table 9 Generated Traffic in Industrial Area

	1971	1981	1991	
Cargo Transportation	960	2, 900	4, 800	(vehicles per day)
Attending Office	452	1, 492	2, 682	
Commercial Use	190	580	960	
Total	1, 602	4, 972	8, 442	

(4) Generated Traffic Following the Construction of Residential Zone.

The residential area on the left bank is planned to house 250, 000 people. The residential area is directly adjacent to the industrial zone on the left bank, and for the greater part of the generated traffic would have the starting and terminal point within the district on the left bank area, and yet there should be a certain quantitative amount of traffic connecting with the right bank for such purposes as going to school or work or on business. However, this traffic goes just the reversed way with and can make use of that for work coming from the right bank, which has already be included in the estimated vehicle number. The traffic, therefore, is estimated to be 30% of the owned vehicle number. If there is an counterpart of peoples' flow-in from the right bank, the total generated traffic volume will come up to be 60% of the owned vehicle number.

Table 10 Generated Traffic in Residential Area

	Population	Owned Vehicle Number	Generated Traffic	
1971	50,000	200	120	(vehicles per day)
1981	150,000	1,000	600	
1991	250,000	2,500	1,500	

(5) Through Traffic Volume.

The completion of the new bridge will introduce the through traffic of long-distance cargo transportation. The through traffic volume in a city area can be generally 10% of the total traffic volume in case of small-size city, and 3 or 4% in case of large city. Accordingly as the development of Chittagong City, the ratio is gradually changed, the expected through traffic will be shown as follows: -

Table 11 Long-Distance Traffic

	Through Traffic
1971	270 (vehicles per day)
1981	520
1991	870

(6) Total of Traffic Volume

Table 12 Expected Traffic Volume

Year	1971	1981	1991
Transferred Traffic from Kalurghat Br.	190	740	1,250
Transferred Traffic from Ferry Boat	570	845	1,215
Generated Traffic in Industrial Area	1,602	4,972	8,442
Generated Traffic in Residential Area	120	600	1,500
Long-distance Traffic	270	520	870
Sub-Total	2,752	7,677	13,277
Small Vehicle	1,345	3,430	5,120
Total	4,097	11,107	18,397

2. Bridge Width

2.1. Width of sidewalk

Since the bridge is to be constructed in the town area, the sidewalk must be needed for the foot-passenger. It is considered to be satisfactory that the bridge would be provided two sidewalks of 6 feet in width on both sides.

2.2. Width of roadway

The required number of lane is to be decided by the hourly traffic, which may be reduced from the daily traffic. The proportional ratio of the peak hourly traffic to the average daily traffic is given from the actual examples to be approximately 7.5% to 10%. In such a case of the proposed bridge as the traffic is all concentrated into the only one bridge over the Karnaphuli, it is considered appropriate to take it up to the higher extent of 10%. Consequently, the expected hourly traffic is shown as follows: -

	1971	1981	1991
Hourly Traffic	410	1, 110	1, 840

The basic traffic capacity per hour for a 2-lane road is given to be 2, 000 by the Highway Capacity Manual (U. S. A.), and the pratical traffic capacity should be lowered to be 60% of it. This standard was resulted from the survey on large-size vehicles in U. S. A. In Japan, where the mixed traffic of large and small vehicles is prevailing, the basic traffic capacity of 2, 500 vehicles per hour is in reality applicable. Since it is presumable that the traffic in Chittagong would be similar to that in the Japanese city, it is considered that a 2-lane road can afford the traffic almost enough by 1991.

For the general national highway in Pakistan, the width of the roadway is given by(lane-number x 10 + 2) feet. On the other hand, it was recommended for the 2-lane road 2 x 13 feet by the U. S. survey team, according to their report submitted for the road construction project of East Pakistan. As the proposed bridge is included in the city planning and the mixed traffic is expected on it, it is appropriated that the 2-lane width should be taken for 2 x 13 = 26 feet.

In case of the 4-lane road, the width per lane might be a little decreased in comparison with that for the 2-lane road, and it would be recommendable to take 4 x 12 = 48 feet.

3. Design Specification

3. 1. General Specifications

In designing the bridge structure, the following Japanese specifications are mainly applied with some modifications. There is not much difference among AASHO, Highway Bridge Code for East Pakistan, and the Japanese specifications, and comparisons among them are shown in the Table 13.

(1) Japanese Standard Design Specification for Highway Steel Bridges. (Japan Road Association: June, 1964)

(2) Japanese Standard Design Specification for Welded Steel Highway Bridges. (Japan Road Association: May, 1964)

(3) Japanese Standard Specification for Composite Girder Bridges. (Japan Road Association: June, 1965)

(4) Japanese Standard Design Specification for Reinforced Concrete Highway Bridges. (Japan Road Association: June, 1964)

(5) Japanese Industrial Standard (JIS), "Steel"

3.2 Modification of Specification

(1) Wind load

In Japan, the wind load is taken to be 300 kg/m^2 in designing, which is converted from the wind velocity of 55 m/s ≈ 123 mph. It was requested in Pakistan that the wind velocity should be 130 mph according to the past cyclone. However, because the load of 55 m/s would work out in the final strength of materials with the allowance of 50%, this was decided to be taken in the designing.

(2) Temperature load

The temperature load is 60°C from -10°C to + 50°C in Japan, but this will be made up to be 50°C from +5°C to + 55° C for the proposed bridge.

(3) Earthquake load

Since there has been almost nothing of earthquake taken place in Pakistan, the earthquake load will be made to be 0. 1 for horizontal seismic coefficient. No vertical seismic coefficient will not be regarded in the designing.

Table 13 Comparison among Specifications in Japan, U.S.A. and East Pakistan

	Japanese Standard Design Specification for Steel Highway Bridge	AASHO	Highway Bridge Code for East Pakistan
Live Loading on the Roadway	T-Loading (T-20) $W = 20^{t}$ ($44,100^{\#}$) L-Loading (T-20) $P = 5^{t}/m$ ($3,360^{\#}/ft$) $W = 350^{kg}/m^2$ ($72^{\#}/ft$) Note: The load intensities, P and W, vary with such factors as bridge width and span.	H-Loading (H-20) $W = 40,000^{\#}$ H - Lane Loadin (H-20) $P = \begin{cases} 18,000^{\#} & \text{For Moment} \\ 26,000^{\#} & \text{For Shear} \end{cases}$ $W = 640^{\#}$/foot of Load Lane Note: The load intensities, P and W, vary with such factors as bridge width and span.	Same as AASHO
Live Loading on the Sidewalk	For the design of slab and floor system $500^{kg}/m^2$ ($102^{\#}/ft^2$) For the design of main girder $350^{kg}/m^2$ ($72^{\#}/ft^2$)	For the design of slab and floor system $85^{\#}/ft$ For the design of main girder $0 < L < 25$ $85^{\#}/ft^2$ $26 < L < 100$ $60^{\#}/ft^2$ $100 < L$ $(30 + \frac{3000}{L})$ $(\frac{55-W}{50})$	Same as AASHO
Impact Loading	$i = \frac{20}{50 + L}$ L: in m	$i = \frac{50}{L + 125}$ L: in feet	Same as AASHO

	Japanese Standard Design Specification for Steel Highway Bridge	AASHO	Highway Brido Code for East Pakistan
Wind Loading	For Plate Girders and Two Main Trusses (kg/m) Deck Plate Girders $240+450h \geqq 600$ Through Plate Girders $450h \geqq 600$ Two Main Trusses Loaded Chords: Loaded case $330+450h \geqq 600$; Unloaded case $360+900h \geqq 600$ Unloaded Chords: Loaded case $450h \geqq 300$; Unloaded case $900h \geqq 300$ "h": The height of the plate girder or chord. (unit: m) For General Types Unloaded with liveload: For the effective vertical projected area of the windside superstructure $300^{kg}/m^2$ ($61^{\#}/ft^2$) For the one of the leeside superstructure $150^{kg}/m^2$ ($31^{\#}/ft^2$) Loaded with Live Load: For the effective vertical projected area of the windside superstructure $150^{kg}/m^2$ ($31^{\#}ft^2$) For the one of the leeside superstructure $75^{kg}/m^2$ ($15^{\#}ft^2$) For Live Load at 1.5m above bridge surface $150^{kg}/m$ ($101^{\#}/ft$) Note: The Load, $300^{kg}/m^2$ is equivalent to the wind load with velocity 55m/sec (123 mile/hour) (wind pressure coefficient: 1.6)	For Truss and Arches $75^{\#}/ft^2$ or over $300^{\#}/ft$ For loaded chords over $150^{\#}/ft$ For non-loaded chords For Girders and Beams $50^{\#}/ft^2$ or over $300^{\#}/ft$ Against Live Load at 6ft above the bridge surface $100^{\#}/ft$ Note: Abovementioned wind load is equivalent to the wind with velocity of 100 mile/hour.	On 1-1/2 times the area of the structure as seem in elevation $50^{\#}/ft^2$ Against a moving line load $200^{\#}/ft$
Temperature	-10°C - 50°C $\triangle t = 60°C$	0° F - 120° F $\triangle t = 120°$ F	40° F - 110° F $\triangle t = 70°$ F
Earthquake Loading $E = k R$	R = Dead Load k = Horizontal Seismic Coefficient 0.35 - 0.10 Vertical Seismic Coefficient 0.10	R = Dead Load k = Horizontal Seismic Coefficient 0.02 - 0.06	R = Dead Load + Live Load k = Horizontal Seismic Coefficient 0.05

<table>
<tr><th></th><th>Japanese Standard Design Specification for Steel Highway Bridge</th><th>AASHO</th><th>Highway Bridge Code for East Pakistan</th></tr>
<tr><td>Allowable stress of steel materials</td><td>SS41, SM41
Axial tension $1,400 kg/cm^2 (19,913^{\#}/in^2)$
Axial compression $\begin{cases} 0<L/r \leq 110 & 1300-0.06(L/r)^2 kg/cm^2 \\ 110<L/r & 7,200,000/(L/r)^2 kg/cm^2 \end{cases}$
Tension due to Bending $1,400 kg/cm^2$
Compression due to Bending
Fixed Flanges $1,300 kg/cm^2$
Non Fixed Flanges $1,300 - 0.6 (L/b)^2 kg/cm^2$
Shearing $800 kg/cm^2 (11,379^{\#}/in^2)$

SM50
Axial tesion $1,900 kg/cm^2 (27,024^{\#}/in^2)$
Axial compression $\begin{cases} 0<L/r \leq 90 & 1800-0.11(L/r)^2 kg/cm^2 \\ 90<L/r & 7,200,000(L/r)^2 kg/cm^2 \end{cases}$
Tension due to Bending $1900 kg/cm^2 (27,024^{\#}/in^2)$
Compression due to Bending
Fixed Flanges $1,800 kg/cm^2 (25,602^{\#}/in^2)$
Non-Fixed Flanges $1,800-1.1(L/b)^2 kg/cm^2$
Shearing $1,100 kg/cm^2 (15,646^{\#}/in^2)$</td><td>ASTM-A7, ASTM-A373
Axial tension $18,000^{\#}/in^2$
Axial compression
$18,000^{\#}/in^2$
$L/r \leq 140$ $\begin{cases} \text{Riveted ends,} & 15,000-\frac{1}{4}\frac{L^2}{r^2}\,^{\#}/in^2 \\ \text{Pin ends} & 15,000-\frac{1}{3}\frac{L^2}{r^2}\,^{\#}/in^2 \end{cases}$
Bending tension $18,000^{\#}/in^2$
Bending compression
Fixed Flange $18,000^{\#}/in$
$L/r \leq 30$ $18,000-5\frac{L^2}{b^2}$
Shear $11,000^{\#}/in^2$

ASTM A-242
<table>
<tr><th>Thickness</th><th>$t \leq 3/4''$</th><th>$3/4'' < t \leq 1\frac{1}{2}''$</th><th>$1\frac{1}{2}'' < t < 4''$</th></tr>
<tr><td>Axial tension</td><td>$27,000^{\#}/in^2$</td><td>$24,000^{\#}/in^2$</td><td>$22,000^{\#}/in^2$</td></tr>
<tr><td>Axial compression
$L/r \leq 125$</td><td>$27,000^{\#}/in^2$</td><td>$24,000^{\#}/in^2$</td><td>$22,000^{\#}/in^2$</td></tr>
<tr><td>Riveted ends</td><td>$22,000-0.56(L/r)^2$</td><td>$20,000-0.46(L/r)^2$</td><td>$18,000-0.39(L/r)^2$</td></tr>
<tr><td>Pin ends</td><td>$22,000-0.73(L/r)^2$</td><td>$20,000-0.61(L/r)^2$</td><td>$18,000-0.48(L/r)^2$</td></tr>
<tr><td>Bending tension</td><td>$27,000^{\#}/in^2$</td><td>$24,000^{\#}/in^2$</td><td>$27,000^{\#}/in^2$</td></tr>
<tr><td>Bending compression</td><td></td><td></td><td></td></tr>
<tr><td>Fixed Flange</td><td>$27,000^{\#}/in^2$</td><td>$24,000^{\#}/in^2$</td><td>$22,000^{\#}/in^2$</td></tr>
<tr><td>$L/b \leq 25$</td><td>$27,000-7.5(L/b)^2$</td><td>$24,000-6.67(L/b)^2$</td><td>$22,000-6.11(L/b)^2$</td></tr>
<tr><td>Shear</td><td>$15,000^{\#}/in^2$</td><td>$14,000^{\#}/in^2$</td><td>$12,000^{\#}/in^2$</td></tr>
</table></td><td>Non</td></tr>
<tr><td>Allowable compressive stress of concrete</td><td>σ_{28} $180 kg/cm^2 (2560^{\#}/in^2)$
Axial compression $\frac{\sigma_{28}}{4}$, below $60 kg/cm^2$ $(853^{\#}/in^2)$
Bending compression $\frac{\sigma_{28}}{3}$, below $80 kg/cm^2$ $(1138^{\#}/in^2)$
For composite girder $\frac{\sigma_{28}}{3.5}$, below $100 kg/cm^2$ $(1,422^{\#}/in^2)$</td><td>Bending compression $0.4\sigma_{28}$
$\sigma_{28} = 2000 - 2400^{\#}/in^2$ n = 15
= 2500 - 2900 = 12
= 3000 - 3900 = 10
= 4000 - 4900 = 8
= 5000 or more = 6</td><td>Bending compression $0.330\sigma_{28}$
σ_{28} is same as AASHO</td></tr>
<tr><td>Allowable tensile stress of reinforcing steel</td><td>SR24 (Equivalent to SS41) $1,400 kg/cm^2 (19,913^{\#}/in^2)$
SD30 (Equivalent to SM50) $1,800 kg/cm^2 (25,602^{\#}/in^2)$</td><td>In flexible member and web reinforcement.
Structural Grade $18,000^{\#}/in^2$
Intermediate Hard Rail Steel Grade $20,000^{\#}/in^2$</td><td>Same as AASHO</td></tr>
</table>

Table 14 Required Material for Type A and Type B

Material	Type A					Type B		
	3-span continuous truss	Composite girder	Cantilever R. C. girder	Simple R. C. girder	Total	3-span continuous truss	2-span continuous truss	Total
Steel Material (t)								
Main truss or girder	(460) 616	(712) 832			(1, 172) 1, 448	(920) 1, 232	(172) 209	(1, 092) 1, 441
Slab	(86) 448				(86) 448	(172) 896	(29) 151	(201) 1, 047
Lateral and sway bracing	126	104			230	252	42	294
Others	83	67	27	10	187	160	28	188
Total	(546) 1, 273	(712) 1, 003	27	10	(1, 258) 2, 313	(1, 092) 2, 540	(201) 430	(1, 293) 2, 970
Reinforcing Steel (t)	159	174	173	43	549	319	53	372
Concrete (cu. m)	1, 188	1, 304	1, 048	272	3, 812	2, 376	397	2, 773
Asphalt Pavement (sq. m)	5, 320	4, 568	1, 658	450	11, 996	10, 640	1, 777	12, 417

Remark: Figures in parentheses are for SM50 steel.

CHAPTER VI

STRUCTURAL DESIGN

1. Type of Superstructure

1.1 Bridge Length

From an economical point of view, it is desirable that the bridge length should be made as short as possible, but, in setting the abutments toward the river so far, there would be a danger of its falling due to the possible scouring at the riverbed in future. Therefore, the abutment on the left bank should be constructed inside of the embankment. On the right bank, there is a shoal exists for 2,000 feet in width, that comes out on the water surface in the dry season but submerged in water in the rainy season. According to the Master Plan, the land reclamation would be conducted on the shoal in future for an effective land use. It is also possible that, if a banking were made on the shoal, the approach road could be constructed on it provided with a sufficient measure taken for draining the flood water in the rainy season, then the bridge being able to end at the river side.

1.2 Navigation Clearance

Ocean-going vessels come up to Chaktai, but not so far the bridge construction site. Making reference to the fact that the span is 156 feet long and the clearance above mean water level 25 feet high in the case of the Kalurghat Railway Bridge which is situated about 7 miles up the proposed bridge construction site, the same navigation limit might be taken for the new bridge. However, considered the prospect over the heavy-industrial zone planned to develop between both bridges, it was decided to make the clearance of 40 feet above the mean water level and the navigation channel width of 200 feet. Within this navigation limit, ships of 500 tons and the country boats with tall masts can pass under the bridge. Most of the country boats are 30 feet or less, from draught line to top of the mast. Due to the long bridge length, even the clearance of 40 feet will not require so much of the banking height of approach roads because the bridge lowers the formation levels at the side spans.

According to the survey result, the cross section of riverbed deepens at the central part, which is used at present as a navigation channel. On the other hand, it is in the Master Plan to keep up with the development of factory area on the left bank that a new navigation channel would be made out to the left bank where the flow runs shallow. Further referred to the past changes in the river course, the proposed bridge is designed taking the main span, which permitting navigation of ships under it, for the length of 1,500 - 1,600 feet inclusive of the proposed navigation channel in the Master Plan and the present deepest run of the river.

1.3 Bridge Type

Among bridge type having the spans sufficient for the navigation channel width of 200 feet, the followings might be the objectives of comparative study:

(1) Girder bridge

- a) Continuous box-girder (non-composite)
- b) Box-girder with steel floor-plate (simple, continuous)
- c) Continuous composite girder (I-section, box-section)

(2) Truss bridge

- a) Simple truss bridge
- b) Cantilever truss bridge
- c) Continuous truss bridge

(3) Arch bridge

- a) Arch bridge
- b) Girder bridge with arch
- c) Arch bridge with stiffening girder

(4) Suspension bridge

- a) Stay-guyed girder bridge
- b) Suspension bridge

Among them, the superstructure of type (1) is simple and light in their appearances, giving a wide view to the driver. However, it requires the tall girder height and the high elevation of road surface accordingly, causing much of banking work on the approach road on the left bank, and the increase of the road construction cost. The type (1-a) is simple in structure and its erection work is easy, but the steel weight in use is large. As for the type (1-b), the dead load is decreased by eliminating the floor-slab work, but the steel weight itself becomes heavy. In case of the type (1-c), the steel weight becomes smaller, but the prestressing to be introduced on the floor-slab-concrete will require the lifting and lowering operation of the supporting points and a considerable amount of prestressing steel bars, making complicate the erection work, so that the construction cost will be large after all. Generally, the types (1) are economical for the span which is not so long as in this bridge.

The type (2) requires lots of member to be used, producing a more or less obstacle to the view of automobile drivers, but for the span of 200 - 300 feet, the small amount of steel weight will be needed, costing a comparatively low cost and making a smart appearance, that might be called the best suitable structure. The types (3), can afford a long span with less numbers of piers. In the case of the only one span of these types, it is good enough for an aesthetic standpoint, but a number of structure aligned in line, as in this bridge, will not come out in any beautiful bridge. The type (4-a) represents an unconventional bridge, enabling to make the main span longer. It would be suitable for such span length that is too long for a girder bridge but too short for a suspension bridge. The construction cost will be comparatively large. The type (4-b) is suitable for a very long span. Furthermore, there will be some problems about its resistance against the vibration by the cyclone.

After the study on the abovementioned types, the truss structure was selected for the main span. Many types are available among the truss structure such as simple type, cantilever type and continuous type. A comparative study on them was made as follows:-

(1) The soil exploration revealed that the ground conditions was reliable, and the substructure would be safely constructed against the large horizontal force which might be transmitted from the cantilever or continuous structure.

(2) The simple type will take the larger steel weight than the other types. No difference is made in steel weight between the cantilever type and continuous one.

(3) The cantilever truss is very complicated in its hinge structure.

(4) The continuous truss maintains a high statically indeterminate degree that will work more advantageously against an unexpected accident than the other types. The continuous truss is easily erected by the cantilever method.

After the study as abovementioned, it was decided to take the continuous truss for the main span. The 3-span continuous type was selected in order that the horizontal force to be transmitted to the substructure would not become too large. From the standpoint of appearance, the span was equally arranged and the paralleled chord truss was adopted.

For the approach span which would not be restricted by the navigation limit for the passage of large ships, two types, Type A and Type B, were studied. In Type A, the span length was designed shorter than that of the main span, and simple and cantilever type reinforced concrete girders and simple live-load composite girders were chosen. In Type B, the same structure was used as the main span.

2. Detail of Superstructure

2.1 Design Specification

Design Load :	TL - 20
Bridge Length :	3520 ft. 4 in.

Bridge Width :	Roadway	2 x 13 = 26 ft.
	Sidewalk	2 x 6 = 12 ft.
Longitudinal Curve :		
Type A	Navigation Span	Horizontal
	Left Side Span	3% Straight Slope
	Right Side Span	2% Straight Slope
Type B	Navigation Span	Horizontal
	Left Side Span	3% Straight Slope
	Right Side Span	2.2% Straight Slope
Cross Grade :	Roadway	1.5% Parabolic Slope
	Sidewalk	1.5% Straight Slope
Track Clearance :	Width	27 ft. 6 in.
	Height	14 ft.
Navigation Clearance :	Width	200 ft.
	Height	40 ft. (above M.W.L.)
Horizontal Seismic Coefficient :		0.1 (for dead load)
Variation of Temperature :		50 °C

2.2 Structure

Type A

2 x Simple reinforced concrete girder (2 x 67'7")

\+ 3 x Simple live-load type composite girder (3 x 172')

\+ 2 x Three-span continuous steel truss (2 x 753')

\+ 5 x Simple live-load type composite girder (5 x 172')

\+ 2 x Cantilever reinforced concrete girder (2 x 247' 10")

Type B

4 x Three-Span continuous steel truss (4 x 753')

\+ 1 x Tow-span continuous steel truss (2 x 503')

(1) Three-span continuous steel truss

i) The principal dimensions are as follows:-

Span length : 250' + 250' + 250'

Distance of center to center between main trusses :	30' 2"
Height of main truss :	27' 3"
Number of panels :	8 + 8 + 8

ii) The members of the main truss and the cross and longitudinal girders are fabricated in the shop by means of electric-arc welding and are spliced and connected with rivets in the field.

iii) The upper chords and the lower chords are connected with the upper lateral bracings and lower lateral bracings respectively in order to stand against the lateral load such as earthquake load or wind load. For the purpose of bringing the lateral load working on the upper lateral bracing to the support, a portal bracing is furnished at the end post of each span. These portal bracing has a clearance enough to provide the track clearance.

iv) Cast-steel roller bearings with four rollers are used as the movable bearings so as to permit the large amount of movement.

(2) Simple live-load type composite girder

i) The principal dimensions are as follows:-

Span length :	170'
Number of main girder :	4
Distance of center to center between main girders :	10'
Cross-section of main girder :	I-section
Height of main girder :	8' 2" 7/16

ii) In this type of girder, dead load of the bridge is supported only by the steel girders themselves; while, after the reinforced concrete slab is hardened, the steel girder and reinforced concrete slab resist the live-load as a composite section. On the top surface of the upper flange of the main girders, shear connectors which can resist the horizontal shearing stress between the slab and steel girders, are welded.

iii) Parts of the girders are fabricated in the shop by means of electric arc-welding and connected or spliced with rivets at the site.

iv) Between every main girders, the sway bracings and lower lateral bracings are furnished for the purpose of resisting lateral loads.

v) Bearings with friction plates are used for the support.

(3) Simple reinforced concrete girder

i) The principal dimensions are as follows:-

Span length :	65' 9"
Number of main girder :	2
Distance of center to center between main girders :	18' 8"
Cross-section of main girder :	Box-section
Height of box girder :	5' 7" 1/2
Width of box girder :	8' 6" (center to center of webs)

ii) In proportion to the considerably large spans, two box-sections are used for the main girders. Cross beams and diaphragms are furnished on the supports and 1/3 points of the span.

iii) Concrete should be casted in site. The strength of the concrete should be σ_{28}=3,500 psi. The size of the main reinforcing bars should be 1 1/4 inch in diameter. The stirrups and bent-up reinforcing bars in the webs resist the shearing force.

iv) Bearings with friction plates are used for the support.

(4) Cantilever type reinforced concrete girder

i) The principal dimensions are as follows:-

Span length :	Suspended span	65' 9"
	+ Cantilever span	16' 3"
	+ Anchor span	82' 0"
	+ Cantilever span	16' 3"
	+ Suspended span	65' 9"
Number of main girder :	2	

Distance of center to center between main girders :	18' 8"
Cross-section of main girder :	Box-section
Height of box girder :	5' 7" 1/2
Width of box girder :	8' 6" (center to center of webs)

ii) Two box girders are used for the main girders in proportion to considerably large spans. On the support of the anchor spans, the lower flanges of the two box girders are connected so as to resist the negative bending moment. Cross girders and diaphragms are furnished on the supports and 1/3 points of the suspended span, on the supports of the cantilever span, and on the 1/4 points of the anchor span respectively.

iii) The strength of the concrete is $\sigma 28$=3,500 psi, and it is casted in site. The size of the main reinforcing bars should be 1 1/4 inch in diameter. Stirrups and bent-up reinforcing bars resist the shearing force.

iv) Bearings with the friction plates serve as supports.

(5) Two-span continuous steel truss

i) The principal dimensions are as follows:-

Span length :	250' + 250'
Distance of center to center between main trusses :	30' 2"
Height of main truss :	27' 3"
Number of panels :	8 + 8

ii) The structure is same as that of the three-span continuous steel truss.

2.3 Erection Work

The superstructure would be erected from the both river banks at the same time toward the middle of the river.

(1) Type A Bridge

i) Simple reinforced concrete girder

As the girder is constructed over the land, the staging is set up with Beatty. The metal form is used for the concreting.

ii) Cantilever reinforced concrete girder

The erection of the spans over the land would be made in the same staging method as that of the simple reinforced concrete girder. For the spans over the water, the staging method is not applicable but the erection truss method. The erection trusses would be supported by the pier and the wooden piles driven in the middle of bridge span.

iii) Simple live-load composite girder

The erection would be carried out in the bent method. Two steel bents are to be used for one span. The members of girder will be transported by boats and the carrier extended between the towers built on the piers. The camber is adjusted, and the riveting is completed, then the bents should be removed.

iv) Three-span continuous steel truss

For the first span, the bent is to be sent up at each panel, and as in the same way as the simple live-load composite girder, the members are brought down by the carrier. When the first span is erected, the second span is set to erect on assembling the members to extend one after another toward the middle, and a bent will be set up in the middle of span to support the extended members. After the erection is completed, the bents and carriers are to be taken out.

(2) Type B Bridge

The first bridge on the left and right banks would be erected in the same steps as in the 3-span continuous truss of Type A. To erect the second bridges from both banks, the tower and carrier will be moved to the next positions.

The upper and lower chords of the first bridge will be connected with the corresponding ones of the second bridge by means of the connecting members for the erection purpose, and the extended in the same way as set forth in the above-mentioned type. When the first span of the second bridge is completed, the connecting members should be removed, and further the extension will be continued.

2.4 Comparison Between Type A and Type B

For the detailed comparisons between Type A and Type B, the following Table 14 should be referred.

Type B structure requires 28% as much steel materials as Type A because the superstructure in Type B is entirely made of steel, while it saves 27% of concrete amount and 32% of reinforcing bars used in Type A structure.

The construction cost for the superstructure including the transportation as well as erection cost is much expensive in Type B. This, however, does not apply to the total construction cost for the bridge including the both superstructure and substructure, because there is less substructure work to be done in Type B. Besides, the percentage of the foreign currency included in the total construction cost should be taken into account in this estimation.

Type A structure, consisting of small span concrete girders on both sides, a little larger span plate girders on the inner sides and large span continuous trusses at the center, displays a beautiful form with a slender appearance emphasized at the center. Type B structure, consisting of the parallel trusses of fourteen equal spans, has beauty of simplicity. It also presents much modern beauty of line as well as light impression. In the both types, the trusses are connected each other with false members to give an effect of entire continuous appearance. In relation to the beauty of the form, both types, having their own merits and becoming to the surrounding scenery as well, are said to match equally.

3. Type of Substructure

3.1 Geological Feature

The ground in the construction site is formed of a thickly accumulated alluvium, and no diluvial stratum did make an appearance even at the depth of 100 m. The alluvium stratum consists of the alternation of fine- and medium-sand and silt, and that there is only a layer of gravel to be seen about at the depth of 40 m. The N-value of the standard penetration test is large enough to show 30 at the depth of 10 m and 40 at 20 m in the sand stratum. It has eventually proved the sand stratum could stand an adequate vertical bearing power for any type of foundation. The silt stratum shows N = 20, proving a considerable horizontal bearing strength. In the deep part, no stratum exists that causes the secondary compression, and there might be no problem at all about the settlement due to consolidation.

3.2 Depth of Scour

From the standpoint of river hydraulics, the scouring on the riverbed would present the most important problem. In carrying out the river training works, it is expected that the rivercourse should be stabilized a little to the left from the center line of the river (as resulted from the model test of the Hydraulic Research Institute). In the meantime, there would remain much anxiety until the river improvement is completed, that the rivercourse might change largely from year to year, and that the concentrated scouring might be caused around the piers.

The probable scouring depth of the planned bridge was estimated as 33 feet below M. S. L. according to the Laursen's formula and Lacy's formula and taking into consideration the example of the Kalurghat Bridge at the same time.

(1) Kalurghat Bridge

Each pier of the Kalurghat Bridge is made of 5 cast-iron pipe-piles of 4 feet 6 inches in diameter. The total width of the pier is 19 feet. At the time of construction, the depth of foundation into the riverbed was 55 feet and the elevation of the riberbed was I. S. L. - 13.75 feet (M. S. L. - 18.24 feet).

The present elevation of the riverbed is not known, but the original foundation depth would be satisfactory for the stability of bridge. The riprap work which was carried out for the thickness of 1.5 feet with stones of about 100 lb. each, seems to be being carried out when necessary.

(2) Discharge and Flood Level

The discharge observation has been continued from 1935 at the Rangamati station. The largest discharge was recorded as 233,000 cfs on 12 July 1946, when the discharge at Chittagong City was presumedly increased to 256,000 cfs. The following is the result of the probability calculation based on the annual maximum discharges for the past 10 years recorded in the report of the Hydraulic Research Station.

Probability	Discharge at Rangamati	Discharge at Chittagong
1/20	250,000 cfs	275,000 cfs
1/50	280,000	308,000
1/100	300,000	330,000

It is stated in the same report that the regulated discharge of the Kaptai Dam is prescribed to be 150,000 cfs in spite of the spillway capacity is at the maximum 562,000 cfs. Therefore, the design flood discharge is taken for 330,000 cfs at the proposed bridge construction site.

The recorded highest tidal level of the Turner & Morrison Compound Tidal Gauge is M. S. L. + 11.93 feet. The report recommends that the highest tidal level should be taken I. S. L. + 17.5 feet (= M. S. L. + 12.01 feet). Therefore, the design flood level is taken for M. S. L. + 12.00 feet.

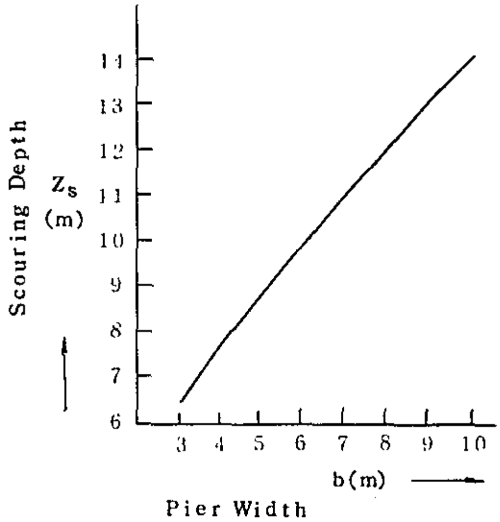
(3) Estimated Depth with Laursen's Formula

The Laursen's formula was resulted from the experiments, and the scouring depth is to be given by the pier width, uniform flow depth, and the shape of pier. The hydraulic characteristics at the proposed construction site are as follows:-

Design Flood Discharge:	330,000 cfs = 11,650 m^3/S	
Cross-sectional Area of Flow:	7,500 m^2	(According to River Survey)
Hydraulic Radius:	7.50 m	(")
Riverbed Slope:	1:20,000	(According to Data of WAPDA)
Coefficient of Roughness:	0.022	(According to River Survey)

With these data, the uniform flow depth is given as 9.38 m. In accordance with the Laursen's formula, the scouring depth is given as shown in the following *figure - 5.*

In case of the front head of pier being made to be in the shape of oval, the scouring depth will be decreased to be 75 - 80% of the value given on the curve. Now, if b = 7.5 m for the pier width, Zs = 12.0 m is given on the figure. Multiplied by the coefficient of shape, given is Zs = 9.0 m.



(4) Estimated Depth with Lacy's Formula

On the other hand, the Pakistan Bridge Code stipulates to use the Lacy's formula to calculate a scouring depth (Proc. ASCE, 1960, HY). In accordance with it:

$$ds = 2 \times 0.473\ (Q/f)^{1/3}$$

ds:	Maximum depth of scour below highest flood level (ft)
Q:	Maximum flood discharge (330,000 cfs)
f:	176 $\sqrt{m}$
m:	Mean grade of bed material (mm)

m = 0.76 mm, given from the soil exploration, makes ds = 72 feet. As the highest flood level is M.S.L. + 12 feet and the elevation of riverbed is M.S.L. - 21 feet, the scouring depth Zs is finally given:

Zs = 72 - (12 +21) = 39 feet (11.7 m)

3.3 Type of Substructure

The piers of a large span bridge would receive a large horizontal external force. As the scouring would cause some decrease in the horizontal bearing power, the depth of foundations under ground should be deeper. The steel pipe-pile foundation is the most advantageous from the economic viewpoint, but that it has a smaller resistance against the horizontal force when scoured. Eventually the pile foundation is disregarded for the large span and also for spans in the deep stream.

Accordingly, no other way but a pneumatic caisson or open caisson would be able to effect the foundation work for the large span. As for the pneumatic caisson, the pressure should be as large as enough to present the water percolation from the sand layer, which would give a danger to the working men's life. Hence the well (open caisson) would be recommended.

For the economical reason, the steel pipe-piling is accepted to use for the short span girder which requires the smaller horizontal resistance. Around the pile foundation is made the riprap work with rubbles so as to protect against the scouring.

All piers would be made of concrete column in order not to be corroded by the river water containing the salt, and further to keep the uniform appearance.

In protection against the collision of ships, fenders made of steel pipe-piles are set up around the piers.

4. Detail of Substructure

4.1 Well Foundation

The well would be made of reinforced concrete into an oval section equipped with a partition wall in the middle of the inside. The stability has been calculated in the state of being scoured, in accordance with the "Design Standard of Japan Highway Public Corporation". The earth pressure was calculated in accordance with the Coulomb's formula, and the earth bearing strength with the Terzaghi's formula. It was assumed that the well would be supported by the sand stratum which had the

N-value of 60 and the interior friction angle of 30°.

(1) Principal dimension

Depth of well :	76 ft. 6 in. - 131 ft.	
Diameter :	Longer diameter	34 ft. - 38 ft.
	Shorter diameter	25 ft.
Thickness :	Side wall	2 ft. 6 in.
	Partition wall	2 ft.
Calculated maximum earth bearing stress :	Vertical	100 t/m^2
	Horizontal	12 t/m^2

(2) Design condition

Designed strength of concrete:	$\sigma 28$ = 246 kg/cm^2	
Allowable stress of concrete:	Compressive	82 kg/cm^2
	Bearing	70 kg/cm^2
	Shearing	7 kg/cm^2
	Bond	8 kg/cm^2
Allowable stress of reinforcing bar:		1,400 kg/cm^2
Circumferential friction around concrete:		0.2 kg/cm^2
Earthquake coefficient:	Horizontal	0.1
Increasing ratio of allowable stress at the time of earthquake :		30%
Live- and dead-load :	Vertical	2,600 t
	Horizontal	470 t
Allowable bearing stress of earth:	Vertical	200 t/m^2
	Horizontal	44 t/m^2

4.2 Steel Pipe-Pile Foundation

The pile foundation would be used for the pier of the simple reinforced concrete girder of the side spans. To keep the stability against the scouring and to make the horizontal displacement of the top down to the small extent, the steel pipe-pile of 100 feet in length and 4 feet in diameter would be used with the concrete placing inside it. For each pier would be used 8 piles, and the top of them would be connected each other by the concrete base (39 ft. x 25 ft. x 5 in.) to fill up the condition of pile-top being connected. The bottom of the pile is assumed to be borne at the sand stratum of N-value = 60, and the bearing strength of the stratum is given as 600 ton per pile by the Meyerhoff's formula. The stress of the pile is given by the Y. L. Chang's formula.

(1) Principal dimension

Steel pipe-pile:	Length	30 m
	Diameter	1,200 mm
	Thickness	12.7 mm
Geometrical moment of inertia:	Steel pipe	876×10^3 cm^4
	Filled concrete	$1,000 \times 10^3$ cm^4
Calculated maximum stress of pile:	Pile-top connected	650 kg/cm^2
	Pile-top free	1,100 kg/cm^2
Calculated maximum displacement of pile top:	Pile-top connected	12 mm
	Pile-top free	106 mm

(2) Design condition

Allowable stress of steel pipe:	1,400 kg/cm^2	
Allowable bearing stress of earth:	600 ton per pile	
Coefficient of horizontal reaction force of earth :	5 kg/cm^3	
Load (including live- and dead-load):	Vertical	104 ton
	Horizontal	18 ton

4.3 Pier and Abutment

(1) Abutment

As the abutment requires a tall height, it is designed to be a buttress type reinforced concrete abutment from a viewpoint of economy and stability. The dimensions are as follows:-

Height:	35 ft. 7 1/10 in.
Width:	34 ft.
Thickness of wall:	2 ft.

(2) Pier

The pier is designed to be a semi-gravity type reinforced concrete pier which features a smaller disturbance to the river flow, easy construction method and good stabilization. This type of pier has an oval section. In case of the steel-pipe pile foundation, the piers are sometimes made of the piles by extending them alone. For this bridge, however, taken into account the possible corrosion and uniform beauty to be produced, all the piers are to be made out in the same column type. The dimensions are as follows:-

Length :	Top	19 ft. - 20 ft.
	Bottom	21 ft. 5 in. - 25 ft. 6 in.
Width :	Top	6 ft. - 7 ft.
	Bottom	8 ft. 5 in. - 12 ft. 6 in.

4.4 Erection Works

For the Type A bridge are needed 2 abutments and 21 piers. Well foundations would be used for both abutments and 13 piers, and pipe-pile foundations for 8 piers.

(1) Coffering Work for Foundation Construction

The coffering work for pile foundation would be carried out to make one group enclosing 2 piers on the left bank side and another group enclosing 6 piers on the right bank side, by means of steel sheet-pile cofferdam in dry works. The cofferdam of well foundations would be carried out for each foundation by means of constructing an artificial island with steel sheet-piles.

The steel sheet-piles would be driven by a Diesel pile-hammer aboard a pile driving boat. The sand and gravel would be deposited inside the sheet-pile wall by a pump dredger.

(2) Steel Pipe-pile Foundation

The steel pipe-pile would be driven into by vibro-hammers. Inside of the steel pipe-pile would be excavated and placed with reinforced concrete.

(3) Well Foundation

Inside of the well would be excavated by a clamshell-type excavator aboard a boat. The metal form would be used for the concreting of the well. The equipments and materials would be carried by boat from the river side.

(4) Pier and Abutment

The concrete for the piers and abutments would be placed in dry works with the metal form and steel staging, by means of concrete tower aboard a boat.

4.5 Comparison between Type A and Type B

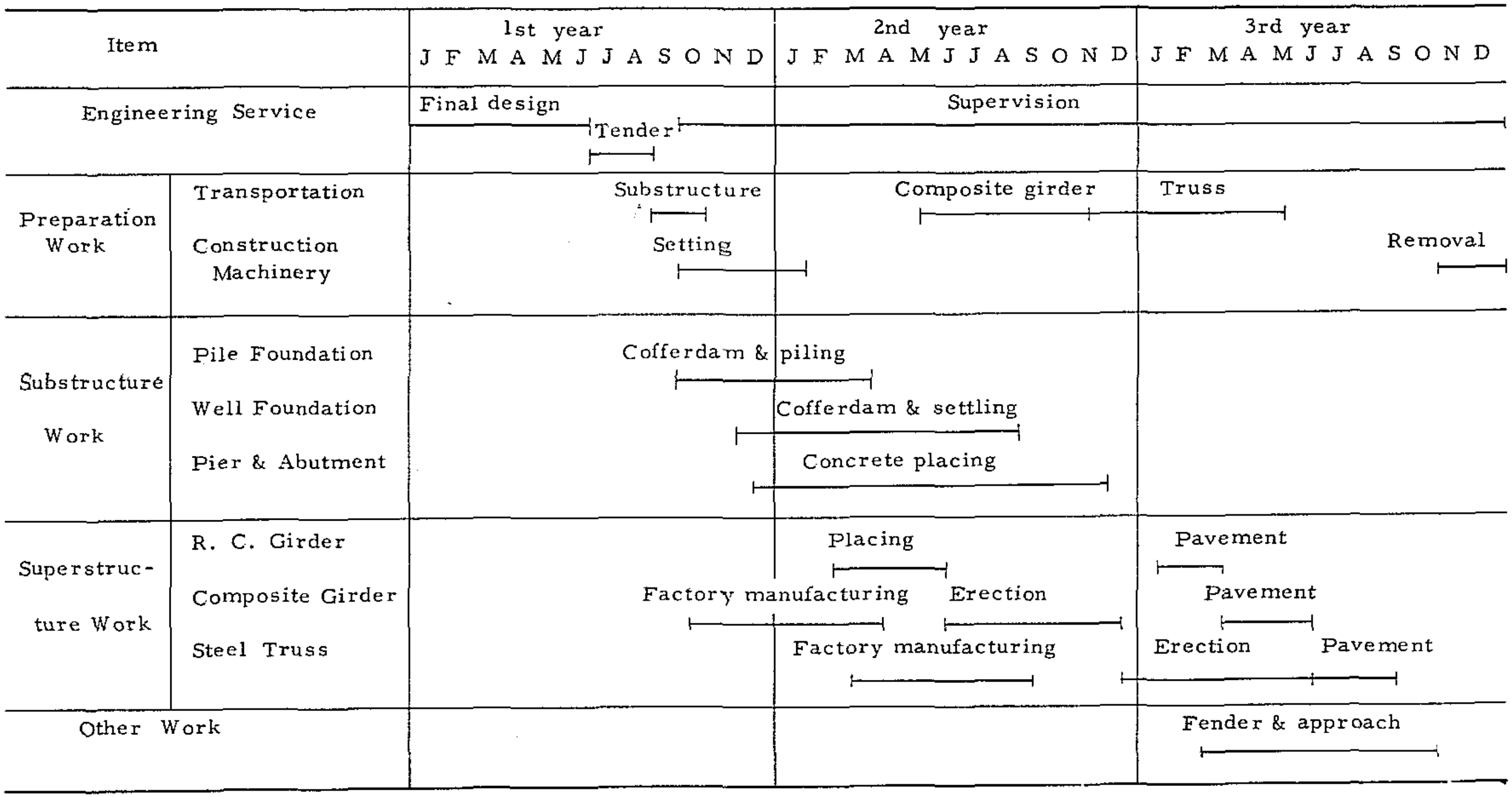
Table 15 Required Material for Type A and Type B

	Material	Type A	Type B
Abutment & pier	Mould frame	95 t	83 t
	Reinforcing bar	191 t	133 t
	Concrete	5,661 m^3	3,962 m^3
Well foundation	Mould frame	650 t	653 t
	Reinforcing bar	1,276 t	1,278 t
	Concrete	20,699 m^3	21,427 m^3
	Excavation	51,483 m^3	50,865 m^3
Pile foundation	Mould frame	7 t	-
	Reinforcing bar	70 t	-
	Steel pipe-pile (ϕ 1200)	635 t	-
	Concrete	2,253 m^3	-
Fender	Steel pipe-pile (ϕ 350)	170 t	105 t
Approach road	Mould frame	29 t	29 t
	Reinforcing bar	72 t	72 t
	Concrete	596 m^3	596 m^3
	Asphalt facing	1,588 m^2	1,696 m^2
Riprap		3,390 m^3	682 m^3

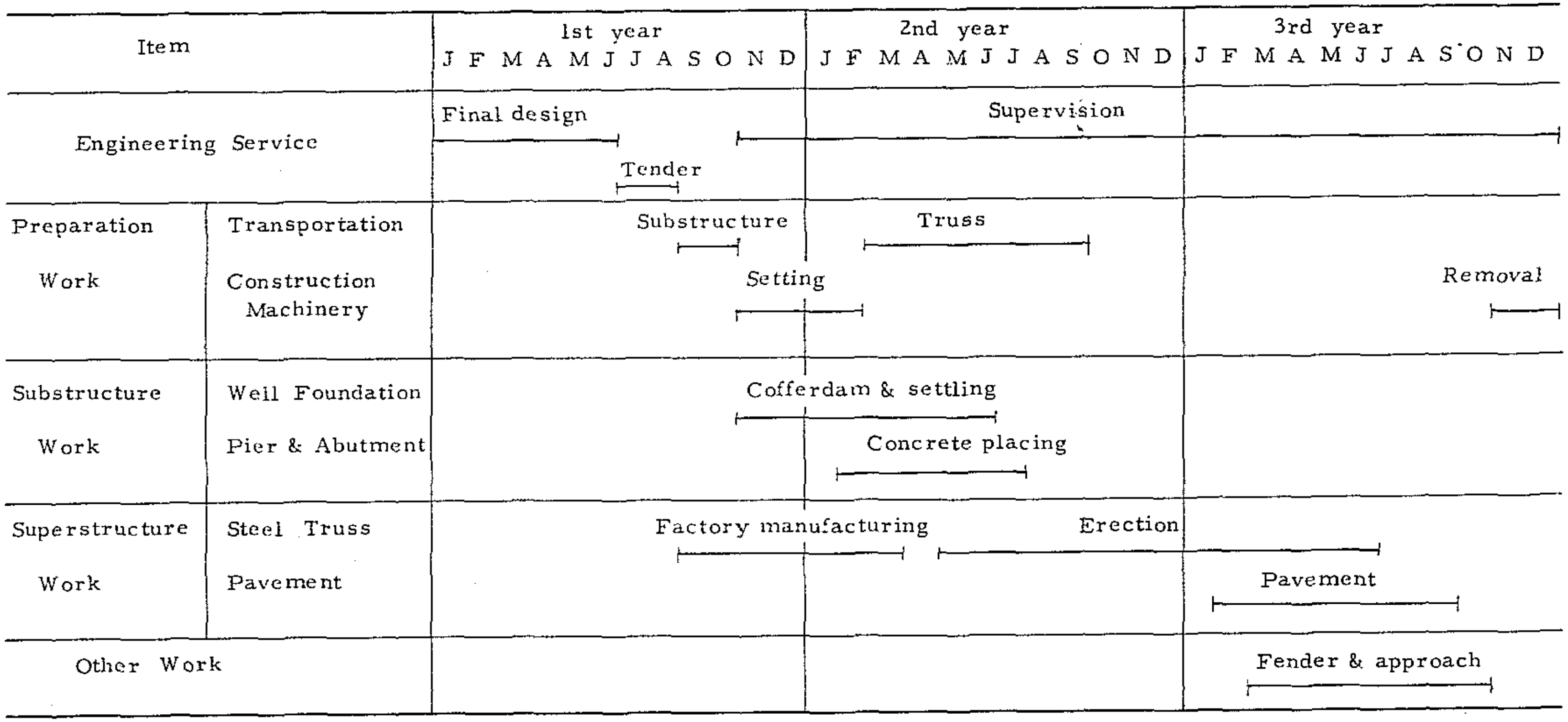
Construction Schedule

The construction work is to start in January 1968. The preparation will take approximately 8 months, including the hydraulic model test and detailed final design. The actual works will be set to begin in September when the rainy season breaks out, and will take 28 months and it is scheduled to complete in December 1970. Of the period, 3 months would be taken for the arrangements of storing place for equipments and materials, transportation of machines, so making the actual working period of 25 months. The construction period should continue in the rainy season.

Time Schedule of Work of Type A Bridge



Time Schedule of Work of Type B Bridge



CHAPTER VII

CONSTRUCTION C T AND FINANCIAL ANALYSIS

1. Construction Cost

1.1 Total Cost

The construction cost is estimated for the 2-lane bridges, Type A and Type B, and for the 4-lane bridges, Type A and Type B. The main dimensions of the 2-lane bridge and 4-lane bridge are as follows:-

	2-lane bridge	4-lane bridge
Length of bridge	3,520 ft. 4 in.	3,520 ft. 4 in.
Effective width	38 ft.	60 ft.
sidewalk	2 x 6 = 12 ft.	2 x 6 = 12 ft.
roadway	2 x 13 = 26 ft.	4 x 12 = 48 ft.
Construction period	3 years	4 years

In this estimation, the approach road and other relative works, which should be carried out being kept a pace with the bridge construction, are not taken into consideration, except the closely adjoining parts of the approach road, of which length is 494 feet on the left bank and 228 feet on the right bank.

The estimated costs are as follows:-

Table 16 Construction Cost US$

	2-lane bridge		4-lane bridge	
	Type A	Type B	Type A	Type B
Field cost	4,150,600	4,221,200	6,255,000	6,361,300
Investigations	83,000	83,000	83,000	83,000
Engineering	290,000	296,000	438,000	446,000
Other	79,000	84,000	85,000	91,000
Total	4,602,600	4,684,200	6,861,000	6,981,300

(1) Field cost

The details of the field cost of the 2-lane bridges are shown in the Table-17 and the following explanation.

(2) Cost of investigations

This is the cost for the soil exploration and the hydraulic model test. The soil exploration should be carried out at the site of all the abutment and pier foundations. The hydraulic model test should be conducted by means of movable bed in order to assume the change in the main flow and the partial scouring.

(3) Engineering fee

This expense is for the engineering services such as the detailed design for bidding, supervision of construction works and necessary testing and inspection.

(4) Other cost

This cost includes expenses for land acquisition and miscellaneous works.

1.2 Field Cost

In calculating the field cost, the following conditions were taken into consideration:-

i) The actual construction work should be executed by Pakistani contractors, and the foreign contractor should only take charge of giving the technical guidance to them.

ii) The required construction period includes 8 months for final designing, bidding and preparing the construction, and 28 months for transportation of materials and equipments and erection work, making a total of 36 months.

The contents of the estimation items are shown as follows:-

Tabel 17-1 Field Cost of Type A Bridge

US$

Item		Construction Equipment		Substructure Work		Superstructure Work		Total		
		Foreign Currency	Local Currency	Foreign Currency	Local Currency	Foreign Currency	Local Currency	Foreign Currency	Local Currency	Total
Equipment Cost	Construction Machinery	77,750		111,070		79,830	1,370	268,650	1,370	270,020
	Accommodation		27,780				12,410		40,190	40,190
	Other Facilities	7,250	8,690			19,000	35,790	26,250	44,480	70,730
	Sub-total									380,940
Material Cost	Steel Material					1,026,630		1,026,630		1,026,630
	Reinforcing Steel			165,390		61,600		226,990		226,990
	Steel Pipe-Pile			122,980				122,980		122,980
	Mould Frame			65,090		36,550		101,640		101,640
	Steel Sheet-Pile	55,560						55,560		55,560
	Cement				243,950		47,450		291,400	291,400
	Sand & Gravel				405,480		67,890		473,370	473,370
	Others			720	55,820	12,690	13,640	13,410	69,460	82,870
	Sub-total									2,381,440
Labour Cost				21,080	120,870	75,040	128,470	96,120	281,490	377,610
Power Cost					16,870		18,010		47,210	47,210
Transportation Cost		26,690		55,300	6,300	195,150	12,210	307,140	18,510	325,650
Fee for Contractor		21,750	8,050	73,970	84,790	195,610	33,150	291,330	125,990	417,320
Tax			15,400		57,420		147,610		220,430	220,430
Total		189,000	104,400	645,600	991,500	1,702,100	518,000	2,536,700	1,613,900	4,150,600
		293,400		1,637,100		2,220,100				

Table 17-2 Field Cost of Type B Bridge

US$

Item		Construction Equipment		Substructure Work		Superstructure Work		Total		
		Foreign Currency	Local Currency	Foreign Currency	Local Currency	Foreign Currency	Local Currency	Foreign Currency	Local Currency	Total
Equipment Cost	Construction Machinery	37,160		109,250		98,650	1,410	245,060	1,410	246,470
	Accommodation		27,780				17,850		45,630	45,630
	Other Facilities	7,250	6,940			19,000	48,680	26,250	55,620	81,870
	Sub-total									373,970
Material Cost	Steel Material					1,359,100		1,359,100		1,359,100
	Reinforcing Steel			152,450		41,330		193,780		193,780
	Steel Pipe-Pile			16,040				16,040		16,040
	Mould Frame			64,090		29,650		93,740		93,740
	Steel Sheet-Pile	53,840						53,840		53,840
	Cement				219,680		34,490		254,170	254,170
	Sand & Gravel				361,390		52,250		413,640	413,640
	Others			720	14,440	15,900	4,460	16,620	18,900	35,520
	Sub-total									2,419,830
Labour Cost			62,870	16,400	109,090	100,540	141,090	116,940	313,050	429,990
Power Cost			13,510		16,880		19,170		49,560	49,560
Transportation Cost		26,650	2,130	64,480	4,710	174,790	10,160	265,920	17,000	282,920
Fee for Contractor		16,200	10,570	54,770	71,430	239,040	32,880	310,010	114,880	424,890
Tax			11,200		43,480		185,360		240,040	240,040
Total		141,100	135,000	478,200	841,100	2,078,000	547,800	2,697,300	1,523,900	4,221,200
		276,100		1,319,300		2,625,800				

Table 18 Detail of Pay Item

	Foreign Currency	Pakistani Currency
Material cost	Steel materials and electric appliances	Concrete materials, oil fuel and wooden materials
Labour cost	Half of perdiem of the foreign engineers	Half of perdiem of the foreign engineers, and total amount of labour cost for the employees in Pakistan
Machinery equipment cost	Depreciation cost for construction machines	Depreciation cost for machines for pavement work
Transportation cost	Charges between Yokohama and Chittagong	Charges between Chittagong port and the construction site
Administrative expences	13% of other cost	10% of other cost (to be paid for the Pakistani contractor fee)
Tax Import duty		10% of the cost for machinery equipments and materials payable in the foreign currency
Income tax		3% of labour cost payable in the foreign currency
Corporation tax		1.2% of the total cost excluding the taxes and administrative cost

Table 19 Construction Machine

	Name	Capacity	Quantity
Substructure	Pile-driving boat	D - 22	6
	Crane boat	30 ton	2
	Tag boat	90 PS	3
	Tripod derrick crane	30 HP	5
	Winch	15 HP	4
	Pump dredger	500 HP	2
	Clamshell	0.6 m^3	5
	Compressor	100 HP	4
	Draining pump	ϕ180 mm	10
	Soil carrier boat	200 m^3	4
	Dump truck	7.5 ton	10
	Vibro-hammer		5
Concrete placing	Batcher plant	2 x 28 Cu. y.	2
	Winch for concrete tower	50 HP	6
	Compressor	75 HP	2
	Dump truck	7.5 ton	4
Steel girder erection	Gate-type steel tower		4
	Carrier	12 ton load	4
	Winch	50 HP	4
	Compressor	50 HP	2
Pavement work	Asphalt plant		2
	Road roller		2

2. Financial Analysis

2.1 Expenditure

(1) Interest

The construction work is to be commenced in January 1968, and completed in December 1970 in case of the 2-lane bridge and in December 1971 in case of the 4-lane bridge.

The borrowings should be obtained at the beginning of the each year in the following proportion to the total cost:-

	2-lane bridge	4-lane bridge
1st year	25%	20%
2nd year	50%	30%
3rd year	25%	30%
4th year	-	20%

The interest rate of the borrowings is assumed 6% per annum.

(2) Maintenance and Repair Expense

The yearly expenditures are estimated as follows:-

Table 20 Maintenance and Repair Expense US$

	2-lane bridge	4-lane bridge
Maintenance & repair expenses		
Road surface of bridge	350	550
Surface of approach road	1,040	1,640
Sideslope of approach road	170	170
Painting	4,920	7,660
Riprap work	1,500	1,500
Illumination	4,490	7,090
Others	1,870	2,790
Sub-total	14,340	21,400
Administrative expenses	9,000	10,800
Total	23,340	32,200

2.2 Revenue

The toll rate is assumed as follows:-

Table 21	Toll Rate
Truck & Bus	¢ 30
Car	¢ 20
Auto Rickshaw	¢ 10
Pedestrian	¢ 2

The toll is collected from the pedestrian lest the ferryboat owner now at work should get into trouble.

The following Table-22 shows the yearly income estimated from the assumed traffic volume and toll rate. The annual operation rate of the road is assumed 90% (329 days per annum).

2.3 Repayment Schedule

According to the repayment schedule (Table-23) based on the aforementioned data, repayment is completed in 1984, that is 17 years after the start of the work, or 14 years after the completion.

In case of a four-lane bridge, however, that repayment takes 21 years from the completion of the work. Moreover, it takes 9 years from the commencement of the work to make profits.

Table 22 Future Traffic Volume & Toll Income

Year	Daily Traffic Volume					Daily Toll Income (US$)						Yearly Income (US$)
	Bus	Truck	Car	Auto-rick-show	Pedes-train	Bus ¢30	Truck ¢30	Car ¢20	Auto-rickshow ¢10	Pedes-train ¢2	Total	
1971	751	1347	654	1345	1700	225	404	131	135	34	929	305,641
2	870	1620	760	1570	1780	261	486	152	157	36	1,092	359,268
3	970	1900	880	1800	1860	291	570	176	180	37	1,254	412,566
4	1075	2150	1010	2040	1940	323	645	202	204	39	1,413	464,877
5	1185	2430	1120	2250	2020	356	729	224	225	40	1,574	517,846
6	1290	1700	1250	2470	2100	387	810	250	247	42	1,736	571,144
7	1400	2950	1380	2670	2180	420	885	276	267	44	1,892	622,468
8	1505	3230	1510	2870	2260	452	969	302	287	45	2,055	676,095
9	1610	3480	1650	3070	2340	483	1044	330	307	47	2,211	727,419
1980	1720	3740	1780	3250	2420	516	1122	356	325	48	2,367	778,743
1	1822	3940	1915	3430	2500	547	1182	383	343	50	2,505	824,145
2	1940	4240	2060	3620	2610	582	1272	412	362	52	2,680	881,720
3	2045	4500	2210	3810	2720	614	1350	442	381	54	2,841	934,689
4	2155	4780	2360	3980	2830	647	1434	472	398	57	3,008	989,632
5	2270	5070	2510	4170	2940	681	1521	502	417	59	3,180	1,046,220
6	2385	5330	2680	4330	3050	716	1599	536	433	61	3,345	1,100,505
7	2490	5620	2830	4480	3160	747	1686	566	448	63	3,510	1,154,790
8	2600	5900	3000	4670	3270	780	1770	600	467	65	3,682	1,211,378
9	2710	6170	3170	4830	3380	813	1851	634	483	68	3,849	1,266,321
1990	2820	6400	3340	5000	3490	846	1920	668	500	70	4,004	1,317,316
1	2942	6795	3540	5120	3600	883	2039	708	512	72	4,214	1,386,406

Table 23-1 Repayment Schedule of 2-lane Bridge

US$

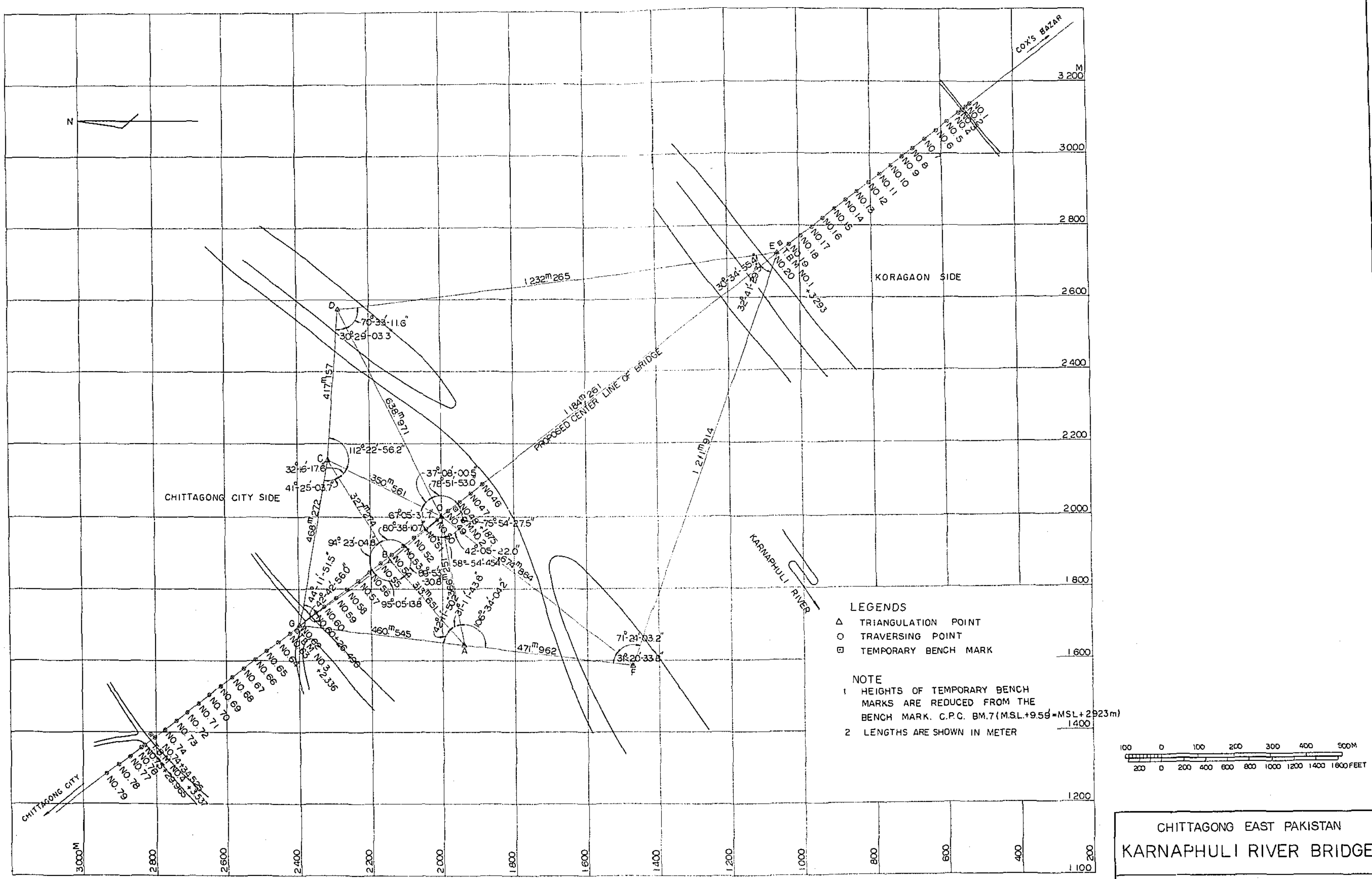
Number of Year	Year	Borrowing	Outstanding (At the beginning of the year)	Expenditures			Revenues	Balance	Accumulated Reserves
				Interests	Maintenance	Total			
1	1968	1,150,650	1,150,650	69,039		69,039		-69,039	
2	9	2,301,300	3,520,989	211,259		211,259		-211,259	
3	1970	1,150,650	4,882,898	292,974		292,974		-292,974	
4	1		5,175,872	310,552	23,340	333,892	305,641	-28,251	
5	2		5,204,123	312,247	"	335,587	359,268	+23,681	
6	3		5,180,442	310,827	"	334,167	412,566	78,399	
7	4		5,102,043	306,123	"	329,463	464,877	135,414	
8	5		4,966,629	297,998	"	321,338	517,846	196,508	
9	6		4,770,121	286,207	"	309,547	571,144	261,597	
10	7		4,508,524	270,511	"	293,851	622,468	328,617	
11	8		4,179,907	250,794	"	274,134	676,095	401,961	
12	9		3,777,946	226,677	"	250,017	727,419	477,402	
13	1980		3,300,544	198,033	"	221,373	778,743	557,370	
14	1		2,743,174	164,590	"	187,930	824,145	636,215	
15	2		2,106,959	126,418	"	149,758	881,720	731,962	
16	3		1,374,997	82,500	"	105,840	934,689	828,849	
17	4		546,148	32,769	"	56,109	989,632	933,523	387,375
18	5		0		"	23,340	1,046,220	1,022,880	1,410,255
19	6				"	"	1,100,505	1,077,165	2,487,420
20	7				"	"	1,154,790	1,131,450	3,618,870
21	8				"	"	1,211,378	1,188,038	4,806,908
22	9				"	"	1,266,321	1,242,981	6,049,889
23	1990				"	"	1,317,316	1,293,976	7,343,865
24	1				"	"	1,386,406	1,363,066	8,706,931

Table 23-2 Repayment Schedule of 4-lane Bridge

US$

Number of Years	Year	Borrowing	Outstanding (At the beginning of the Year)	Expenditures			Revenues	Balance
				Interests	Maintenance	Total		
1	1968	1,372,200	1,372,200	82,332		82,332		-32,332
2	9	2,058,300	3,512,832	210,770		210,770		-210,770
3	1970	2,058,300	5,781,902	346,914		346,914		-346,914
4	1	1,372,200	7,501,016	450,061		450,061		-450,061
5	2		7,951,077	477,065	32,200	509,265	305,641	-203,624
6	3		8,154,701	489,282	"	521,482	359,268	-162,214
7	4		8,316,915	499,015	"	531,215	412,566	-118,649
8	5		8,435,564	506,134	"	538,334	464,877	-73,457
9	6		8,509,021	510,541	"	542,741	517,846	-24,895
10	7		8,533,916	512,035	"	544,235	571,144	26,909
11	8		8,507,007	510,420	"	542,620	622,468	79,848
12	9		8,427,159	505,630	"	537,830	676,095	138,265
13	1980		8,288,894	497,334	"	529,534	727,419	197,885
14	1		8,091,009	485,461	"	517,661	778,743	261,082
15	2		7,829,927	469,796	"	501,996	824,145	322,149
16	3		7,507,778	450,467	"	482,667	881,720	399,053
17	4		7,108,725	426,524	"	458,724	934,689	475,965
18	5		6,632,760	397,966	"	430,166	989,632	559,466
19	6		6,073,294	364,398	"	396,598	1,046,220	649,622
20	7		5,423,672	325,420	"	357,620	1,100,505	742,885
21	8		4,680,787	280,847	"	313,047	1,154,790	841,743
22	9		3,839,044	230,343	"	262,543	1,211,378	948,835
23	1990		2,890,209	173,413	"	205,613	1,266,321	1,060,708
24	1		1,829,501	109,770	"	141,970	1,317,316	1,175,346
25	2		654,155	39,249	"	71,449	1,386,406	1,314,957
26	3		0		"			

DRAWINGS

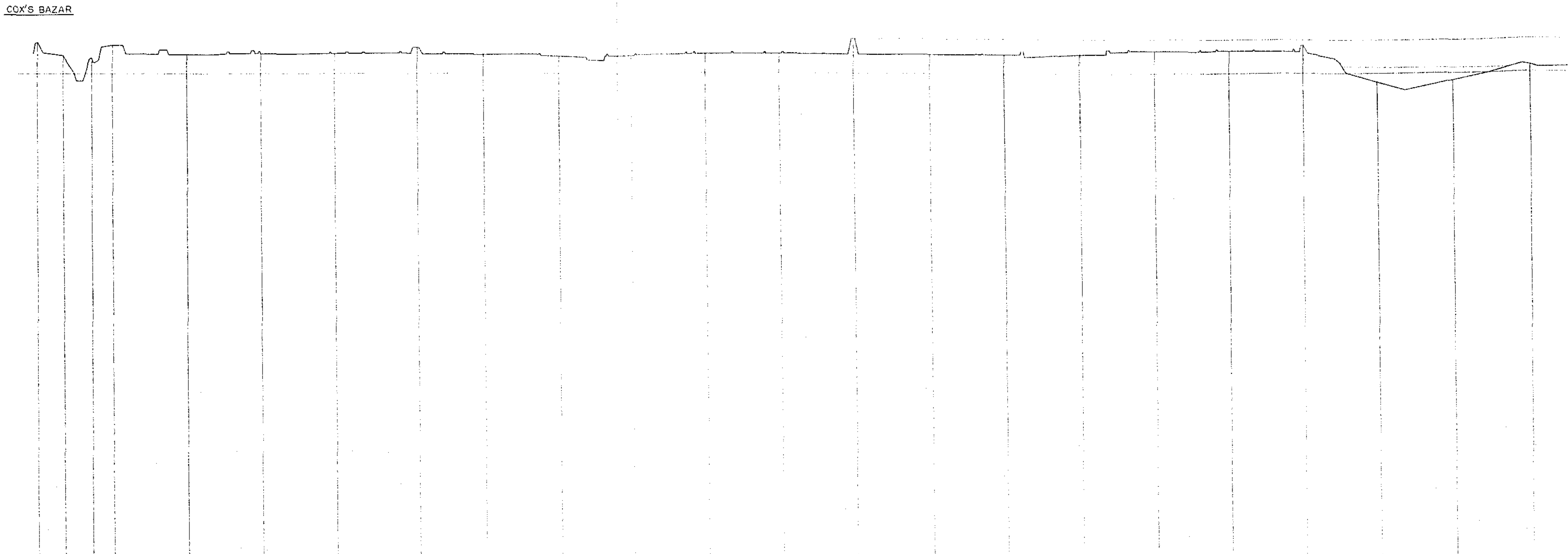


CHITTAGONG EAST PAKISTAN

KARNAPHULI RIVER BRIDGE

NETWORK MAP OF TRIANGULATION AND TRAVERSING POINTS

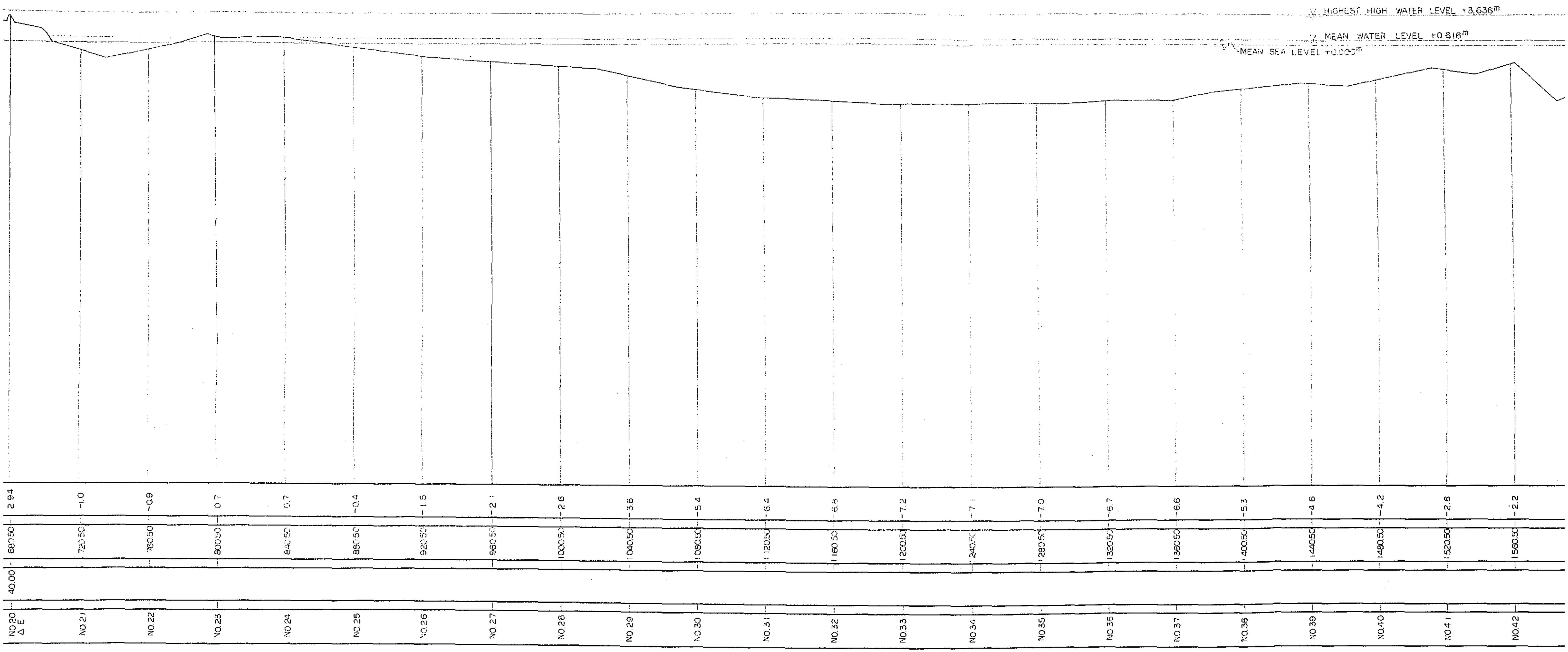
FEB. 1965	I — 1

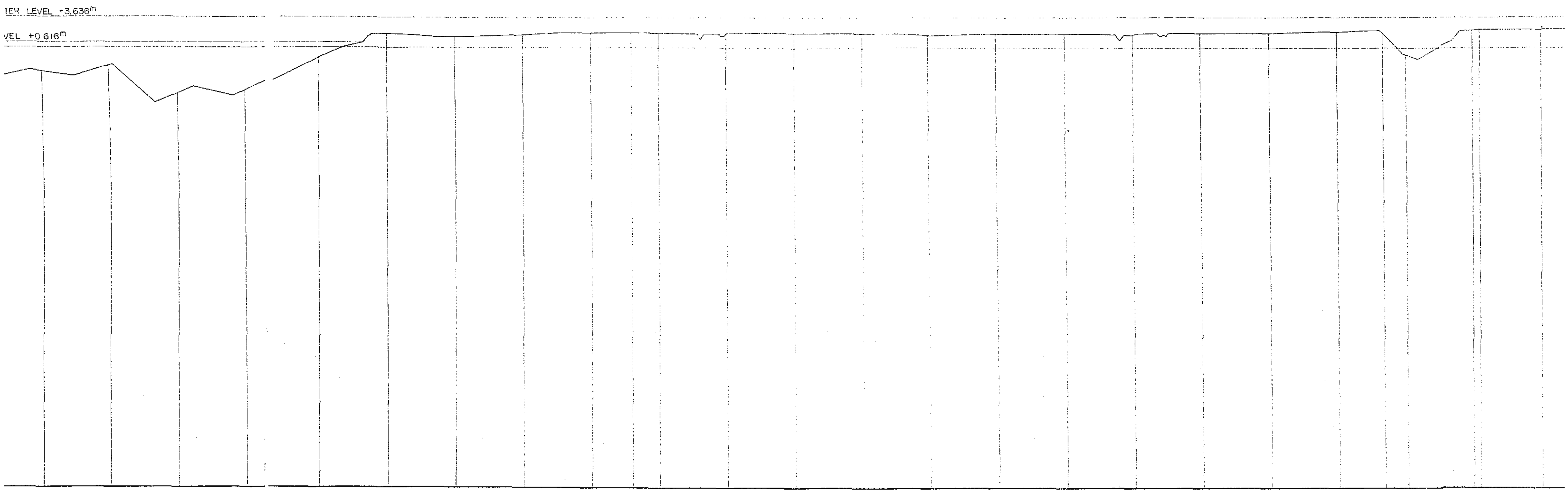


GROUND HEIGHT	3.27	1.91	1.70	3.13	2.04	2.14	2.05	2.73	1.92	1.76	1.83	1.98
ACCUMULA-TIVE DISTANCE	0.00	14.00	29.00	40.50	80.50	120.50	160.50	204.50	240.50	280.50	320.50	360.50
DISTANCE	0.00	14.00	15.00	11.50	40.00	40.00	40.00	44.00	36.00	40.00	40.00	40.00
STATION	NO.1	NO.2	NO.3	NO.4	NO.5	NO.6	NO.7	NO.8	NO.9	NO.10	NO.11	NO.12

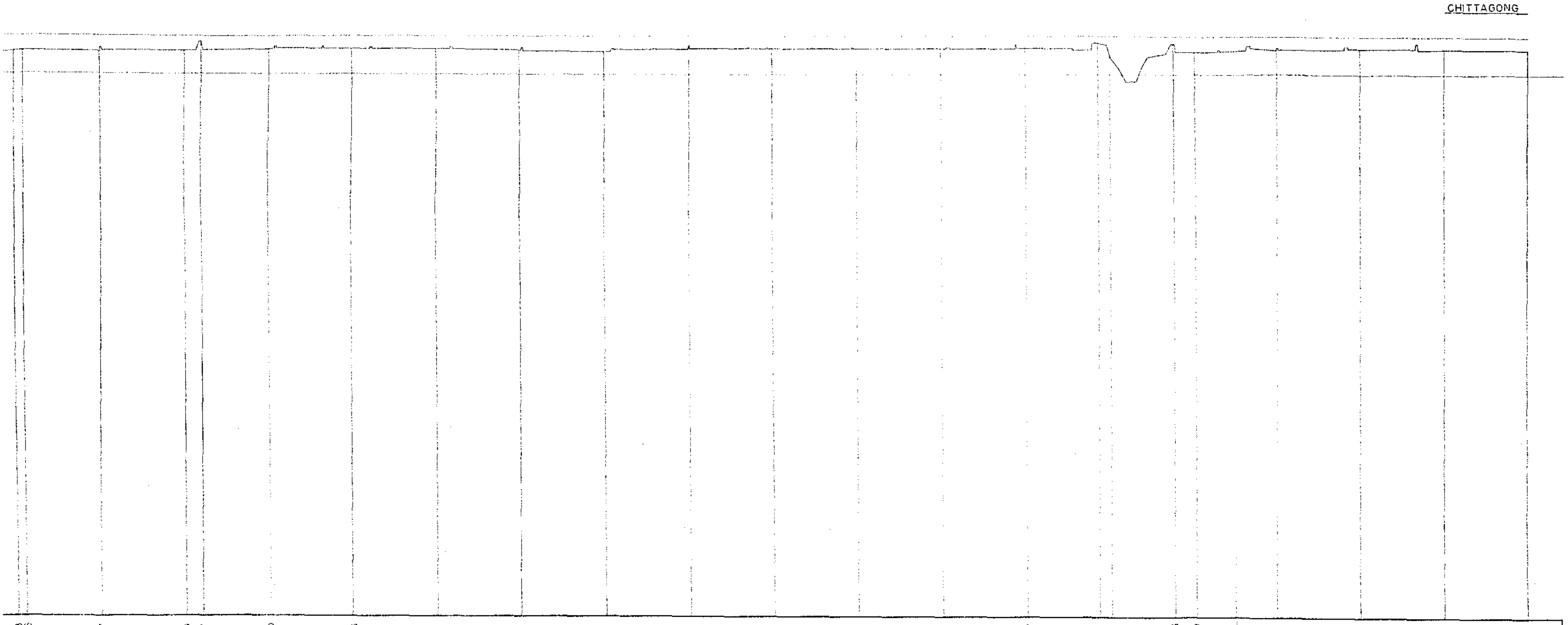
GROUND HEIGHT	1.96	3.58	1.93	1.96	1.96	2.30	2.34	2.94	-1.0	-0.9	0.7
ACCUMULA-TIVE DISTANCE	400.50	439.50	480.50	520.50	560.50	600.50	640.50	680.50	720.50	760.50	800.50
DISTANCE	40.00	39.00	41.00	40.00	40.00	40.00	40.00	40.00			
STATION	NO.13	NO.14	NO.15	NO.16	NO.17	NO.18	NO.19	NO.20 Δ E	NO.21	NO.22	NO.23

MEASURED IN METER

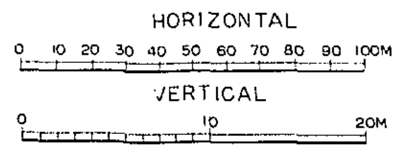

HIGHEST HIGH WATER LEVEL +3.636m
MEAN WATER LEVEL +0.616m
MEAN SEA LEVEL +0.000m
2.94
-1.0
-0.9
0.7
0.7
-0.4
-1.5
-2.1
-2.6
-3.8
-5.4
-6.4
-6.8
-7.2
-7.1
-7.0
-6.7
-6.6
-5.3
-4.6
-4.2
-2.8
-2.2
NO.20
NO.21
NO.22
NO.23
NO.24
NO.25
NO.26
NO.27
NO.28
NO.29
NO.30
NO.31
NO.32
NO.33
NO.34
NO.35
NO.36
NO.37
NO.38
NO.39
NO.40
NO.41
NO.42



NO.41	-2.8	1520.50	
NO.42	-2.2	1560.50	
NO.43	-5.5	1600.50	
NO.44	-5.0	1640.50	
NO.45	-1.2	1680.50	
NO.46	1.62	1720.50	
NO.47	1.30	1760.50	40.00
NO.48	1.52	1800.50	40.00
NO.49	1.74	1840.50	40.00
Δ 0	1.73	1864.761	24.261
NO.50	1.68	1880.50	15.739
NO.51	1.52	1920.50	40.00
NO.52	1.70	1960.50	40.00
NO.53	1.66	2000.50	40.00
NO.54	1.42	2040.50	40.00
NO.55	1.62	2080.50	40.00
NO.56	1.60	2120.50	40.00
NO.57	1.40	2160.50	40.00
NO.58	1.68	2200.50	40.00
NO.59	1.71	2240.50	40.00
NO.60	1.83	2280.50	40.00
NO.60 +26.496	1.50	2306.996	26.496
NO.61		2320.50	13.504
NO.62	2.08	2360.50	40.00
Δ 6	2.16	2364.585	4.085
NO.63	2.07	2400.50	35.915



NO.62 Δ 6	40.00 4.085	2360.50 2364.585	2.03 2.16
NO.63	35.915	2400.50	2.07
NO.64	40.00	2440.50	2.30
NO.64 +7.75	7.75	2448.25	3.07
NO.65	32.25	2480.50	2.40
NO.66	40.00	2520.50	2.46
NO.67	40.00	2560.50	2.45
NO.68	40.00	2600.50	2.38
NO.69	40.00	2640.50	2.17
NO.70	40.00	2680.50	2.43
NO.71	40.00	2720.50	2.41
NO.72	40.00	2760.50	2.39
NO.73	40.00	2800.50	2.37
NO.74	40.00	2840.50	2.50
NO.74 +34.525	34.525	2875.025	2.98
NC 75	5.475	2880.50	
NO.75 +29.965	29.965	2910.465	2.96
NO.76	10.035	2920.50	2.10
NO.77	40.00	2860.50	2.32
NO.78	40.00	3000.50	2.44
NO.79	40.00	3040.50	2.41
NO.80	40.00	3080.50	2.41

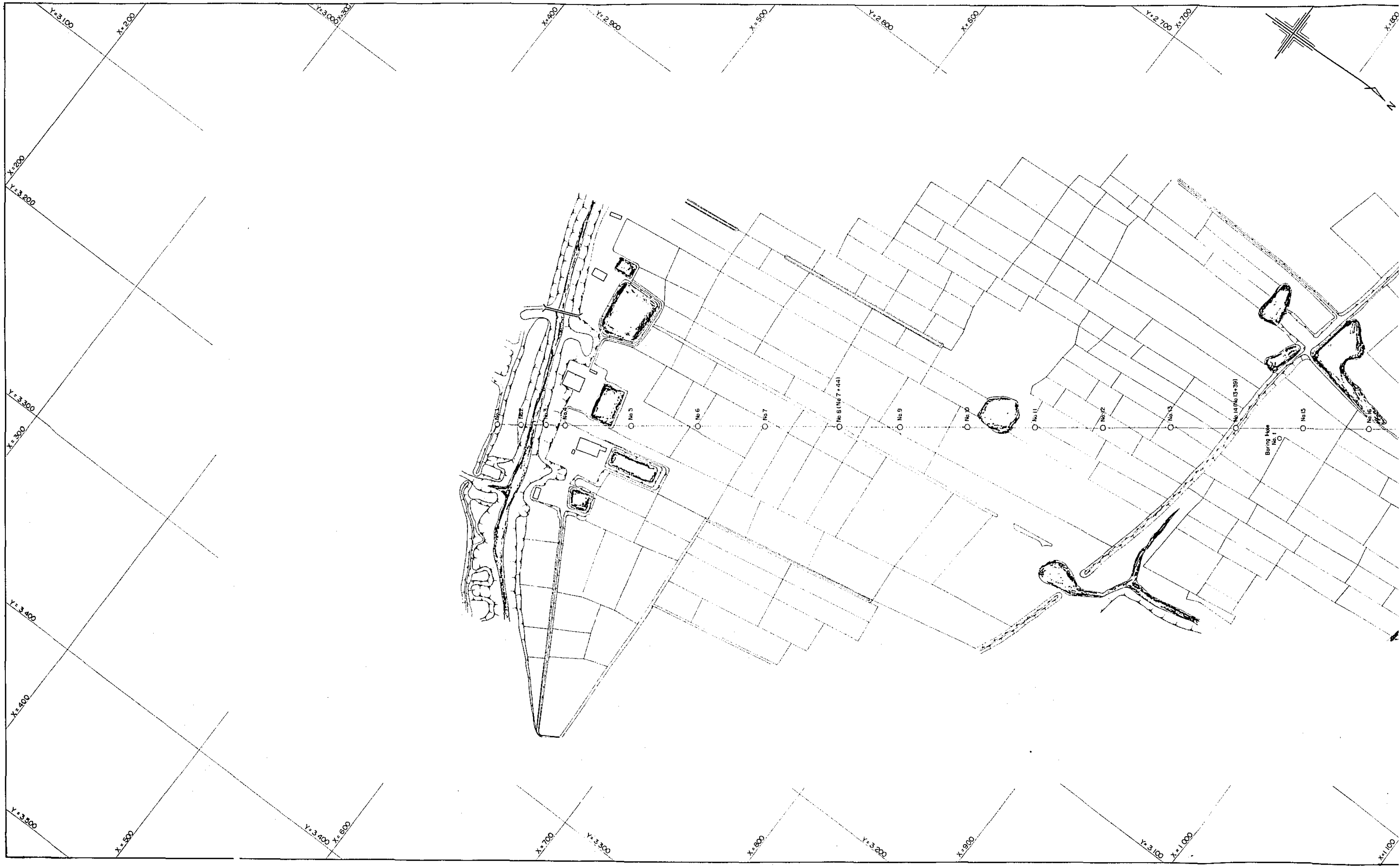


CHITTAGONG EAST PAKISTAN

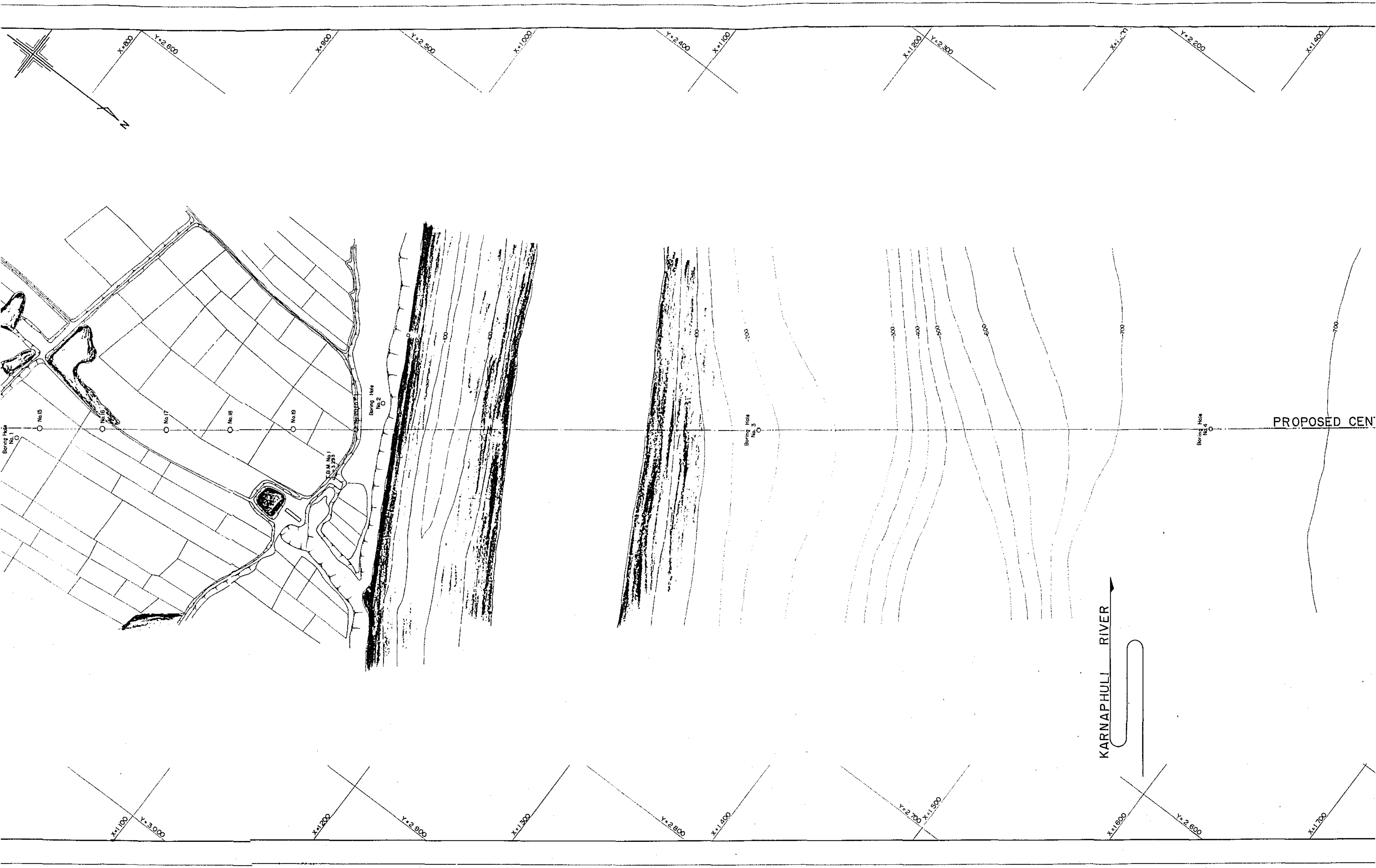
KARNAPHULI RIVER BRIDGE

PROFILE AT PROPOSED BRIDGE CROSSING OVER KARNAPHULI RIVER

FEB. 1965 | I – 2

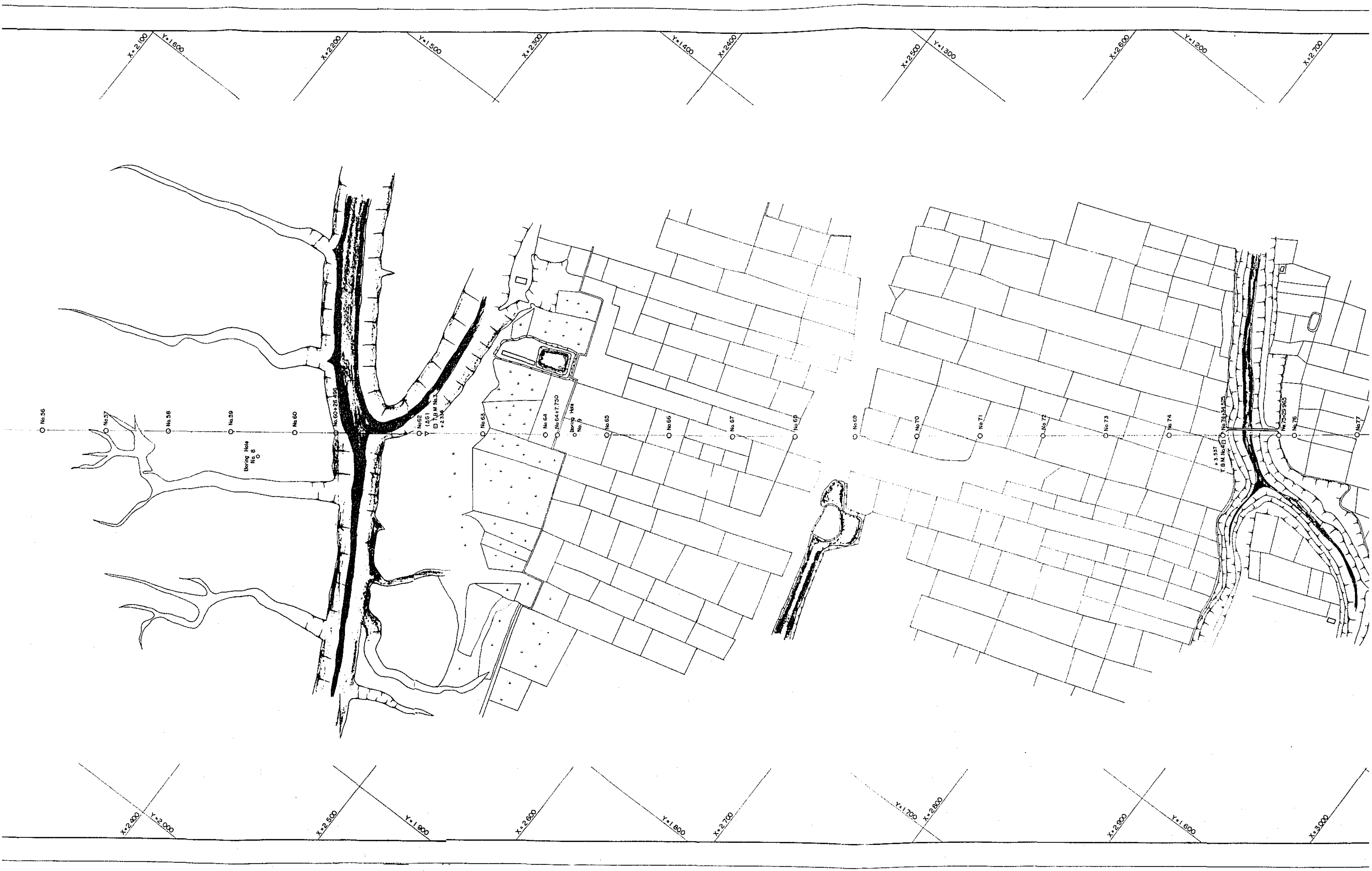


Y=3 100
X=200
Y=3 000
X=400
Y=2 900
X=500
Y=2 800
X=600
Y=2 700
X=700
X=200
Y=3 200
Y=3 300
X=300
Y=3 400
X=400
Y=3 500
X=500
Y=3 400
X=600
X=700
Y=3 300
X=800
Y=3 200
X=900
Y=3 100
X=1 000
X=1 100
N
No.4
No.5
No.6
No.7
No.8 (No.7+44)
No.9
No.10
No.11
No.12
No.13
No.14 (No.13+39)
No.15
No.16
Boring Hole No.1

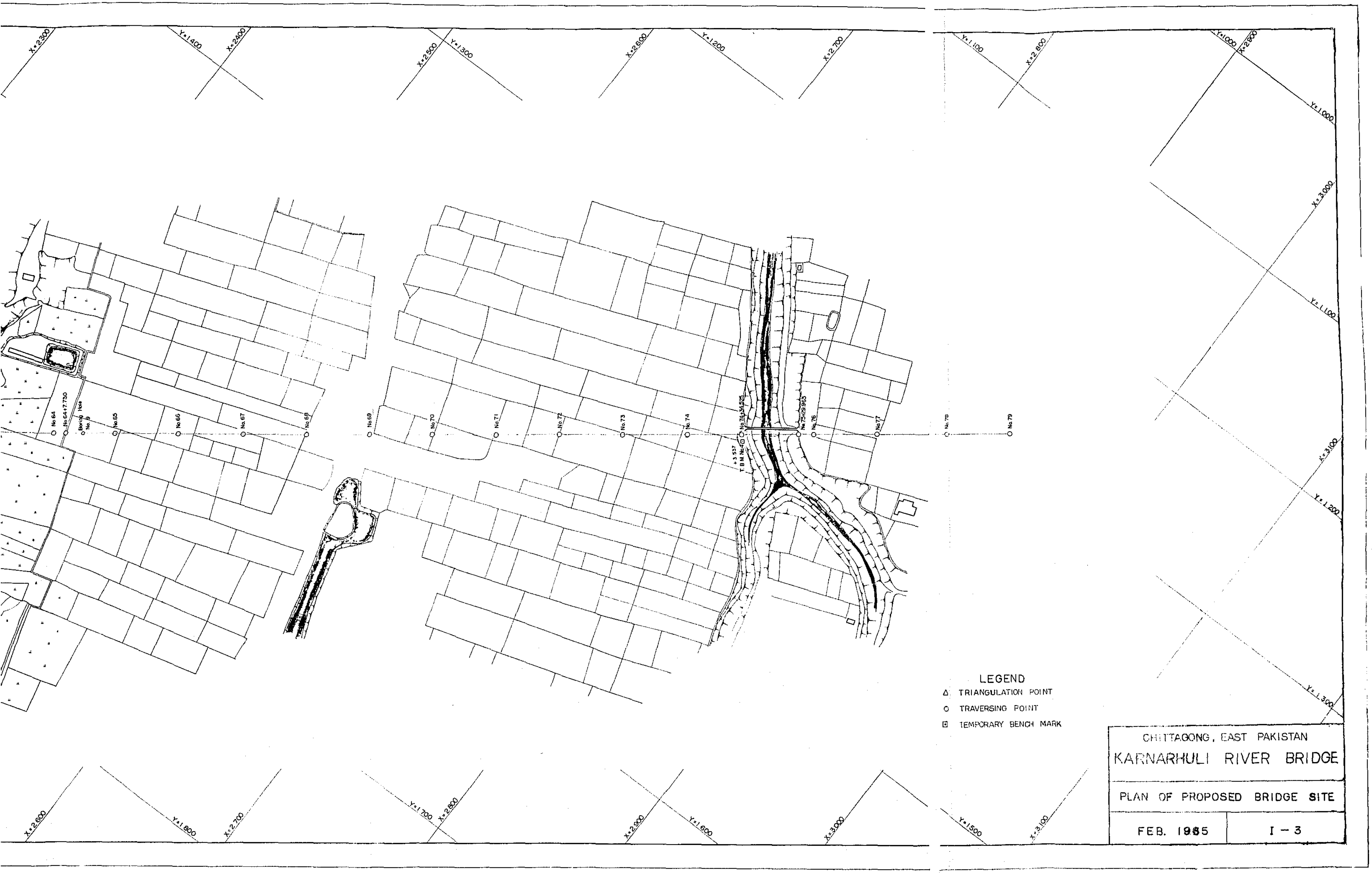


PROPOSED CENT
KARNAPHULI RIVER
Boring Hole No. 1
No.15
No.16
No.17
No.18
No.19
Boring Hole No. 2
Boring Hole No. 3
Boring Hole No. 4
B.M. No.1
X=800
Y=2,600
X=900
Y=2,500
X=1,000
Y=2,400
X=1,100
X=1,200
Y=2,300
Y=2,200
X=1,400
X=1,100
Y=3,000
X=1,200
Y=2,900
X=1,300
Y=2,800
X=1,400
Y=2,700
X=1,500
X=1,600
Y=2,600
X=1,700

POSED CENTER LINE
Boring Hole No.5
Boring Hole No.6
Boring Hole No.7
Water Gauge No.1
Water Gauge No.2
No.46
No.47
No.48
No.49
No.50
No.51
No.52
No.53
No.54
No.55
No.56
T.B.M No.2
MEASURED IN METER
0m
10
20
30
40
50m
100m
150m
200m
X=1,400
Y=2,100
X=1,500
Y=2,000
X=1,600
X=1,700
Y=1,900
X=1,800
Y=1,800
X=1,900
Y=1,700
X=2,000
X=1,700
Y=2,500
X=1,800
Y=2,400
X=1,900
X=2,000
Y=2,300
X=2,100
Y=2,200
X=2,200
Y=2,100
X=2,300



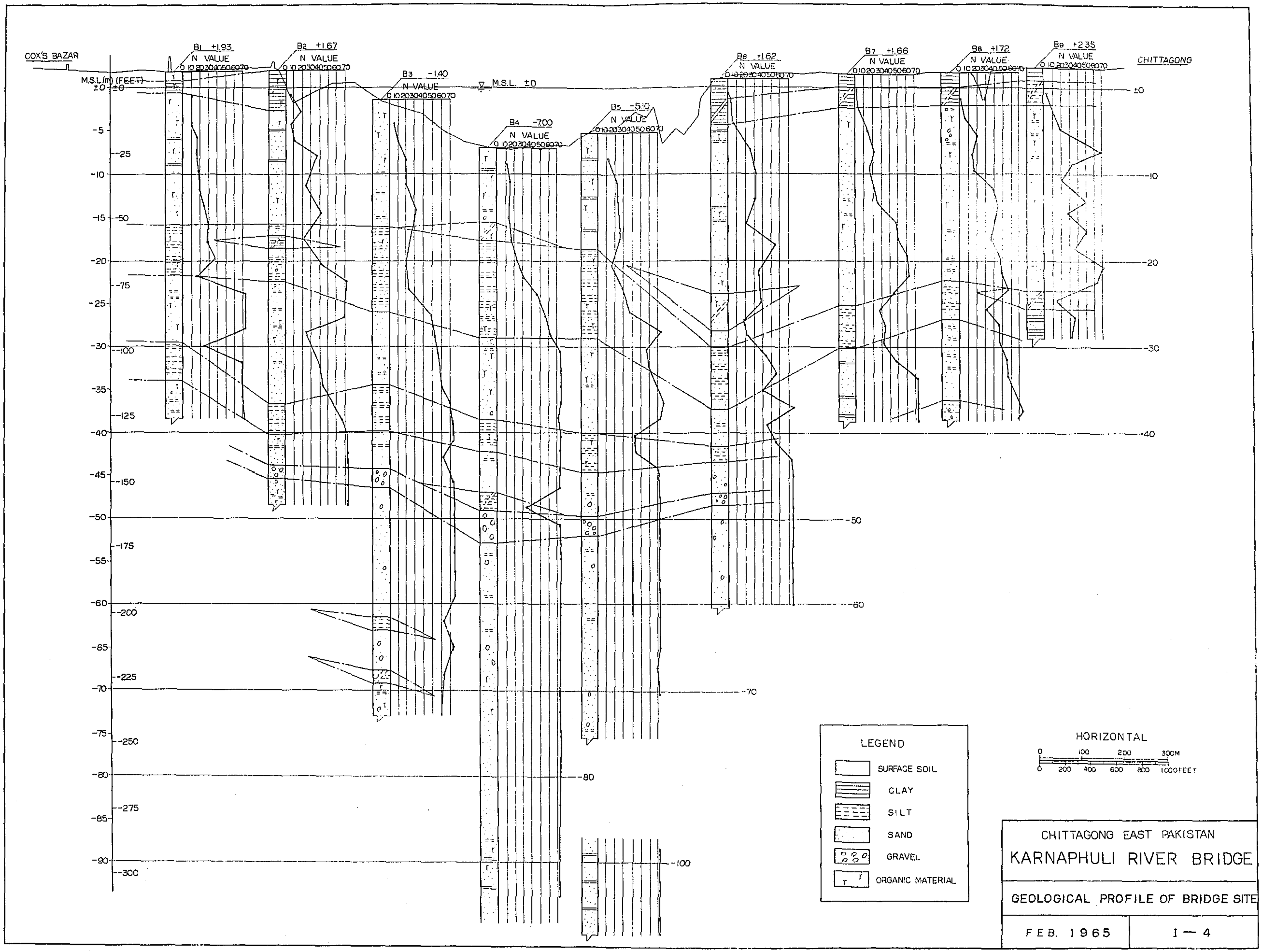
X=2100
Y=1600
X=2200
Y=1500
X=2300
Y=1400
X=2400
X=2500
Y=1300
X=2600
Y=1200
X=2700
No.56
No.57
No.58
No.59
No.60
No.60+26.896
No.62
No.63
No.64
No.64+7.750
Boring Hole No.8
Boring Hole No.9
No.65
No.66
No.67
No.68
No.69
No.70
No.71
No.72
No.73
No.74
T.B.M. No.4
No.75
No.76
No.77
X=2400
Y=2000
X=2500
Y=1900
X=2600
Y=1800
X=2700
Y=1700
X=2800
X=2900
Y=1600
X=3000



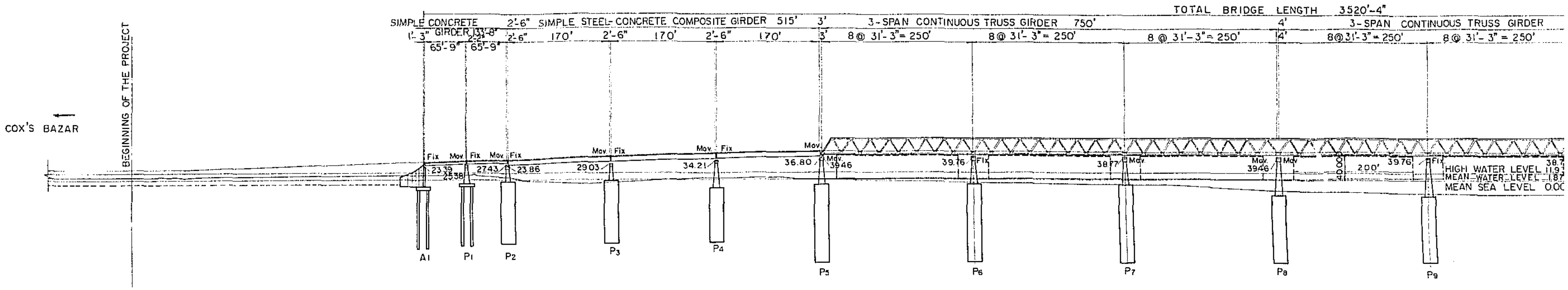
LEGEND

- △ TRIANGULATION POINT
- ○ TRAVERSING POINT
- ⊡ TEMPORARY BENCH MARK

CHITTAGONG, EAST PAKISTAN	
KARNARHULI RIVER BRIDGE	
PLAN OF PROPOSED BRIDGE SITE	
FEB. 1965	I — 3

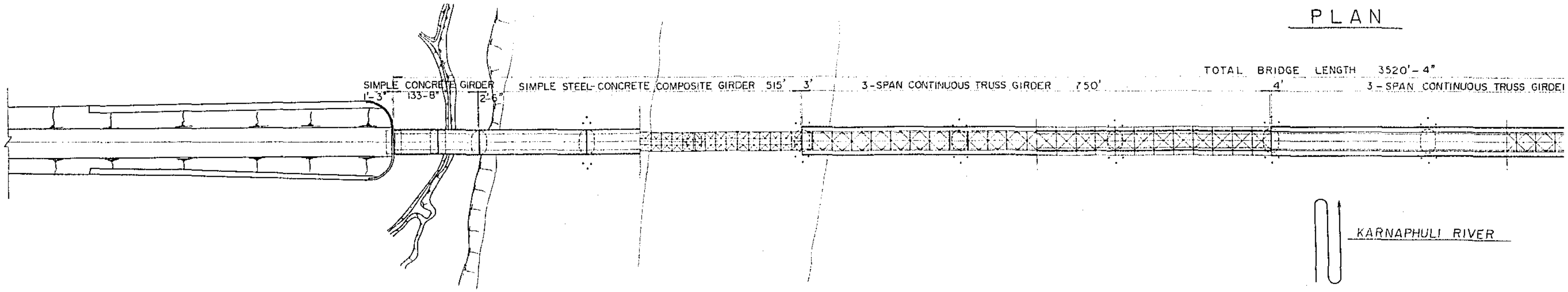

COX'S BAZAR
CHITTAGONG
M.S.L.(m) (FEET)
M.S.L. ±0
B1 +1.93
B2 +1.67
B3 -1.40
B4 -7.00
B5 -5.10
B6 +1.62
B7 +1.66
B8 +1.72
B9 +2.35
N VALUE
LEGEND
SURFACE SOIL
CLAY
SILT
SAND
GRAVEL
ORGANIC MATERIAL
HORIZONTAL
0 100 200 300M
0 200 400 600 800 1000FEET
CHITTAGONG EAST PAKISTAN
KARNAPHULI RIVER BRIDGE
GEOLOGICAL PROFILE OF BRIDGE SITE
FEB. 1965
I — 4

ELEVATION

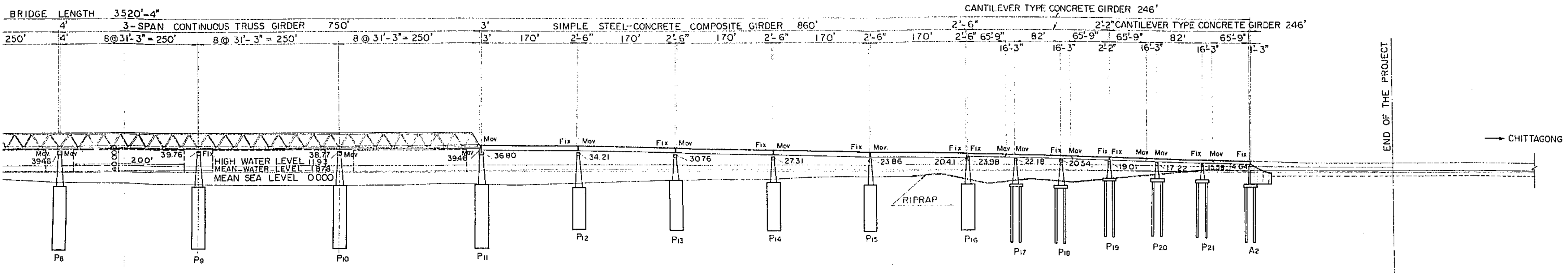


SLORE	LEVEL	PARABOLIC SLOPE						3% STRAIGHT SLOPE			PARABOLIC SLOPE		LEVEL			
PROPOSED HEIGHT	18.CC	18 70	19.18	2057 2096	24.51	28.44	29.89	31.93 32.38	33.97	39.15	4432	4692	46.92	4692	4692	4692
GROUND HEIGHT	6.43	6.43	5.91	6.43 6.54	7.55	7 71	7.61	7.55 9 65	5.25	4.17	2 30	1.64	-5 25	-886	-17.06	-21.98
ACCUMULATIVE DISTANCE	1629320	1707656	1753260	1838888 1851760	1970120	2101.352	2149567	2217650 2232584	2285733	2458233	2630733	2803483	3054983	3304983	3556983	3808983
DISTANCE		78.336	45.604	85.628 12.872	118 360	131.232	48 215	68083 14.934	53 149	172 500	172 500	172 750	251500	250000	252000	252000
STATION		No.16		No.17	No18	No 19	(A1)	(P1) No20	(P2)	(P3)	(P4)	(P5)	(P6)	(P7)	(P8)	(P9)

PLAN

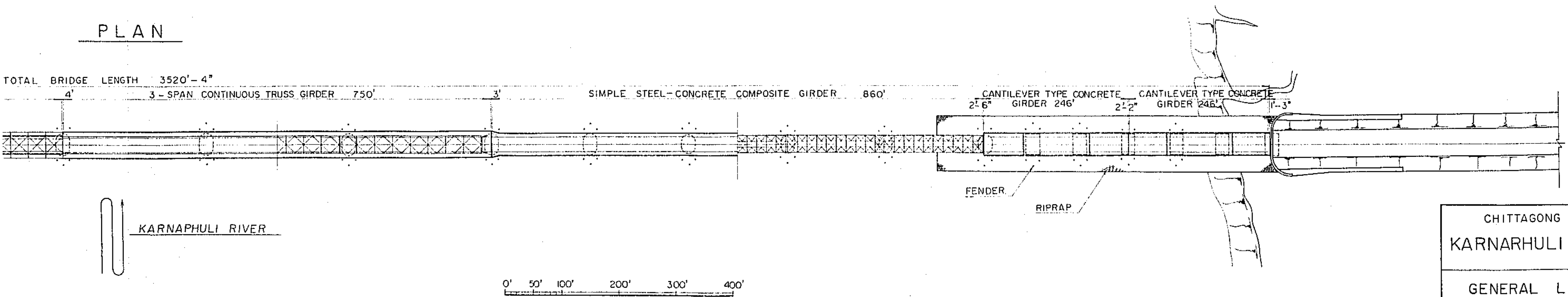


ELEVATION



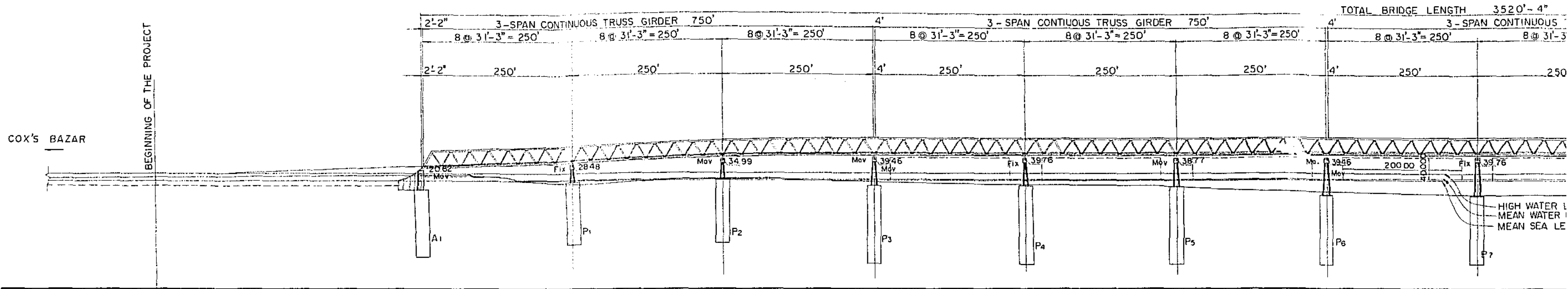
LEVEL / PARABOLIC SLOPE / 20% STRAIGHT SLOPE / PARABOLIC SLOPE / LEVEL				
(P8)	46.92	-17.06	3556983	252000
(P9)	46.92	-21.98	3808983	252000
(P10)	46.92	-23.29	4058983	250000
(P11)	46.92	-22.31	4310483	251.500
(P12)	44.32	-19.68	4483.233	172.750
(P13)	40.87	-14.76	4655733	172.500
(P14)	37.42	-14.76	4828233	172.500
(P15)	33.97	-11.15	5000.733	172.500
(P16)	30.52	-14.76	5173.233	172.500
(P17)	28.86	-18.04	5255483	83.250
(P18)	27.22	-17.39	5338483	82.000
(P19)	25.56	-12.63	5421566	83.083
(P20)	23.90	-4.92	5504649	83.083
(P21)	22.26	1.31	5586649	82000
No46	21.10	5.31	5644615	57966
(A2)	20.59	5.35	5669899	25.284
	19.97	5.03	5700941	31042
No47	19.20	4.27	577 847	74.906
	18.99	4.59	5799445	23.594
No48	18.10	4.99	5907079	86.914
	18.00	5.00	5918635	11.556
No49	18.00	5.71	6038.311	131.232

PLAN



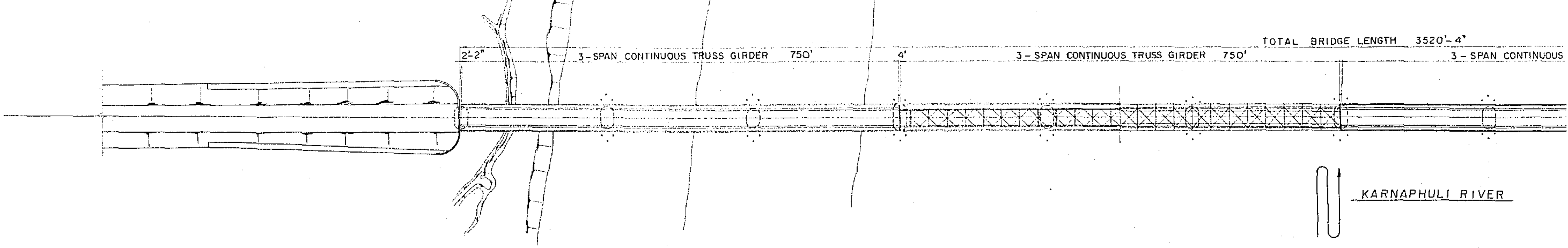
CHITTAGONG EAST PAKISTAN	
KARNARHULI RIVER BRIDGE	
GENERAL LAYOUT	TYPE A
OCT. 1965	II — 1

ELEVATION

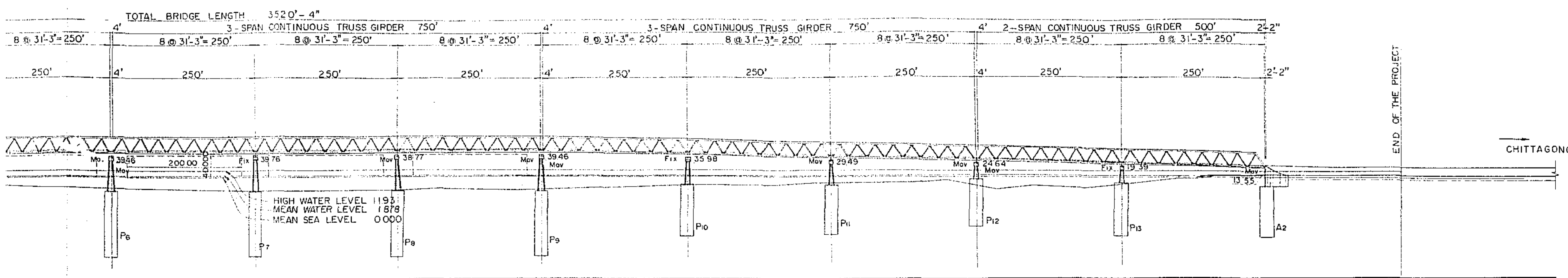


SLOPE	LEVEL	PARABOLIC SLOPE						3% STRAIGHT SLOPE		PARABOLIC SLOPE		LEVEL			
PROPOSED HEIGHT	1800	1985	2141	2171	2269	2662	2207	3056	3564	4314	4692	4692	4692	4692	4692
GROUND HEIGHT	6.43	6.43	6.43	7.30	7.55	7.71	7.61	9.65	-377	1.64	-1.80	-6.89	-11.91	-21.00	-2379
ACCUMULATIVE DISTANCE	1715386	1813886	1838888	1912386	1970120	2101352	2149567	2232584	2401734	2651734	2903734	3155734	3405734	3657734	3909734
DISTANCE		98500	25002	73498	57734	131232	48215	83017	169150	250000	252000	252000	250000	252000	252000
STATION			No 17		No 18	No 19	(A1)	No 20	( )	(P2)	(P3)	(P4)	(P5)	(P6)	(P7)

PLAN

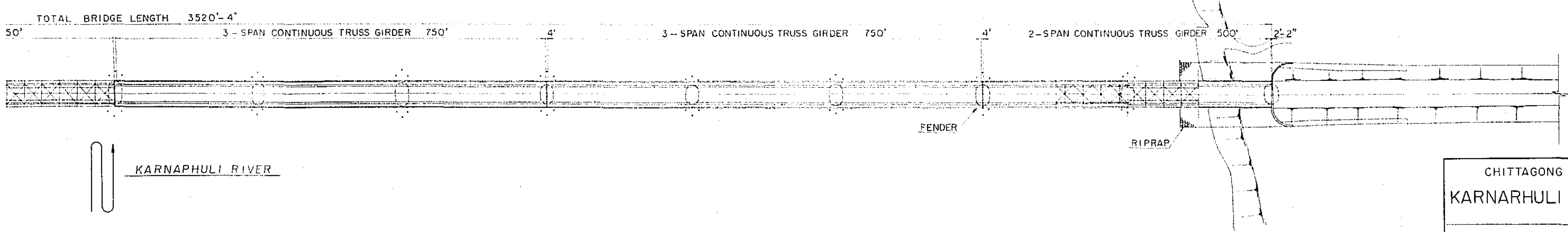


ELEVATION



LEVEL / PARABOLIC SLOPE / 2.2% STRAIGHT SLOPE / PARABOLIC SLOPE / LEVEL				
(P6)	252000	3657734	-2100	4692
(P7)	252000	3909734	-2379	4692
(P8)	250000	4159734	-2329	4692
(P9)	252000	4411734	-21.82	4692
(P10)	252000	4663734	-1608	43.14
(P11)	250000	4913734	-1450	3764
(P12)	252000	5165734	-14.11	3209
(P13)	252000	5417734	-13.12	2655
No46	226881	5644615	5.31	21.56
(A2)	25284	5669899	5.35	2100
	37850	5707749	490	2017
No47	68095	5775847	427	19.41
	30405	5806252	459	1908
No48	98500	5904752	4.99	1800
	2.327	5907079	5.01	1800
No49	131.232	6038311	5.71	18.00

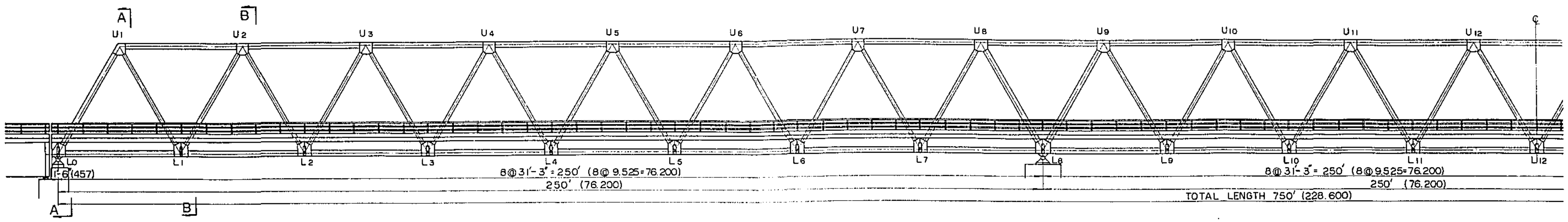
PLAN



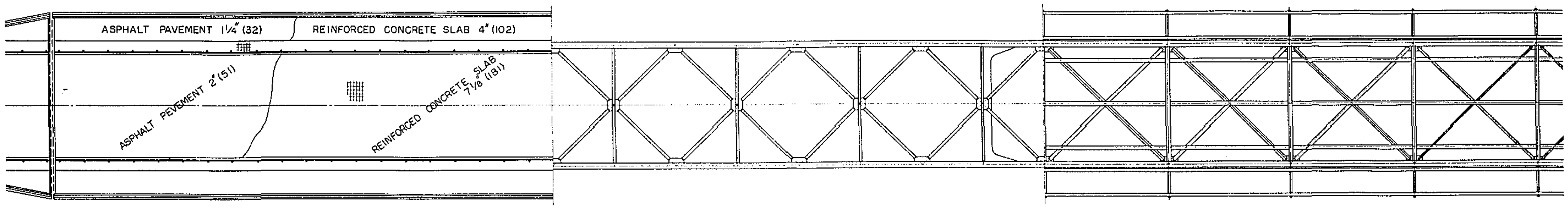
0' 50' 100' 200' 300' 400'

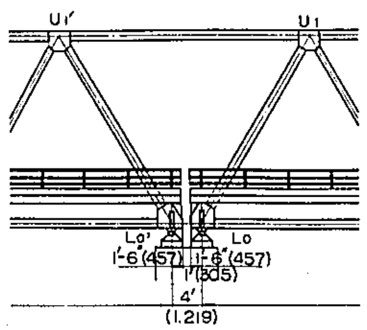
CHITTAGONG EAST PAKISTAN	
KARNARHULI RIVER BRIDGE	
GENERAL LAYOUT TYPE B	
OCT. 1965	II - 2

ELEVATION

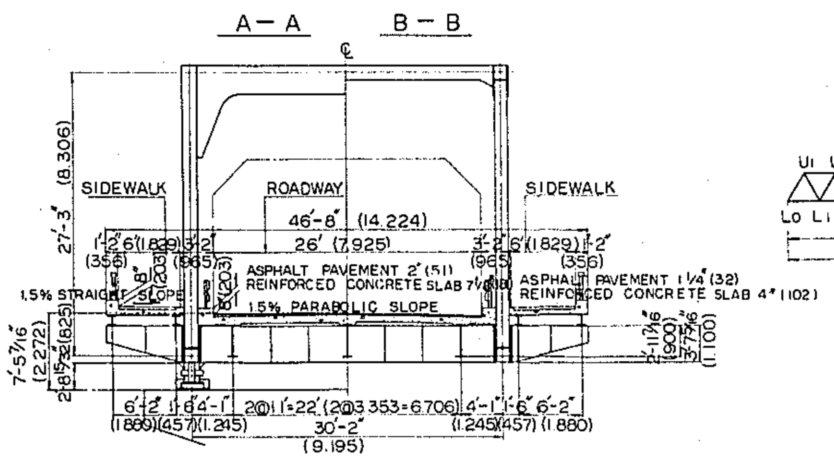


PLAN

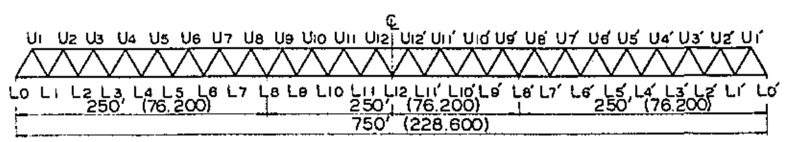




SECTION



KEY PLAN

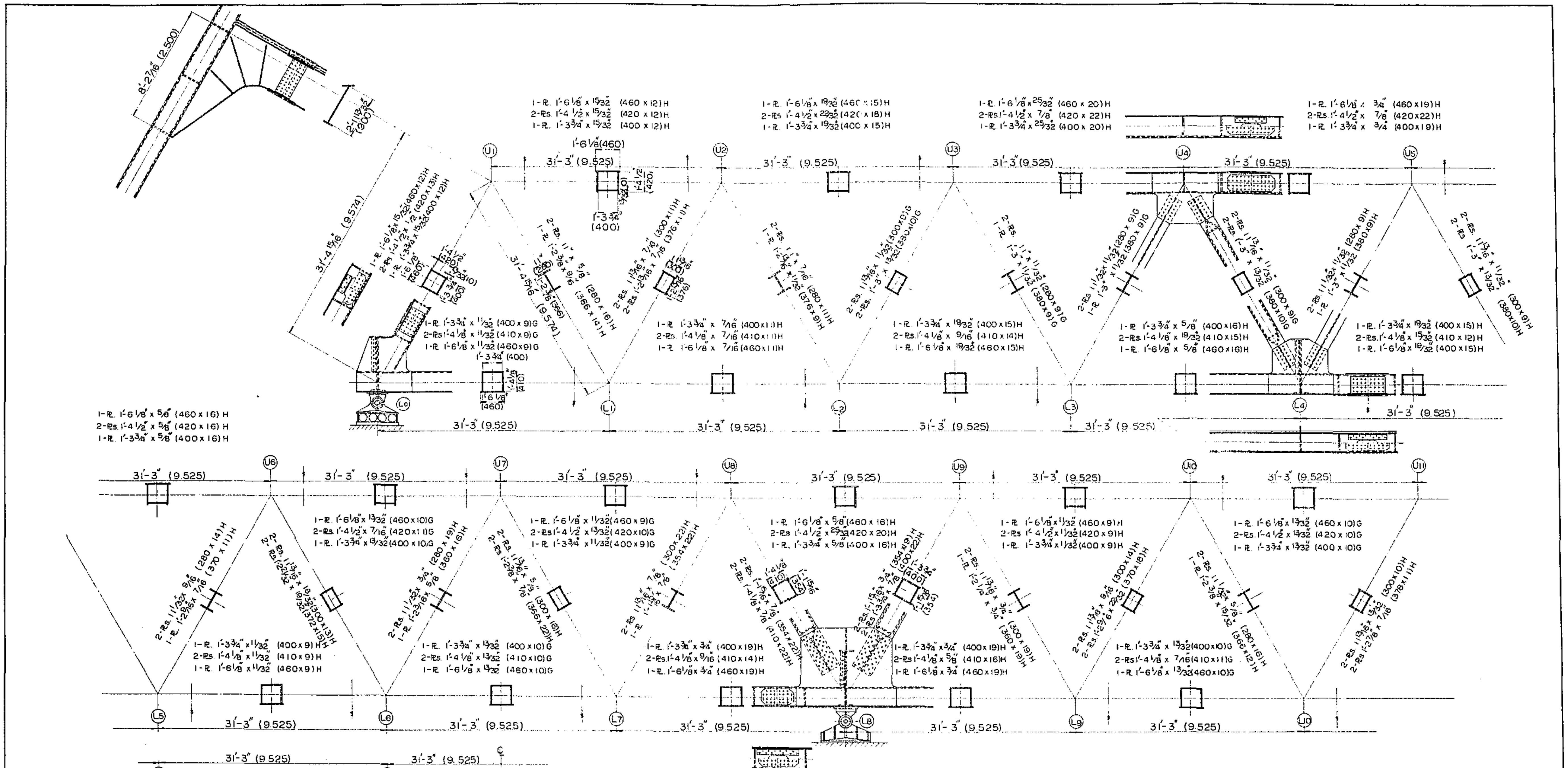


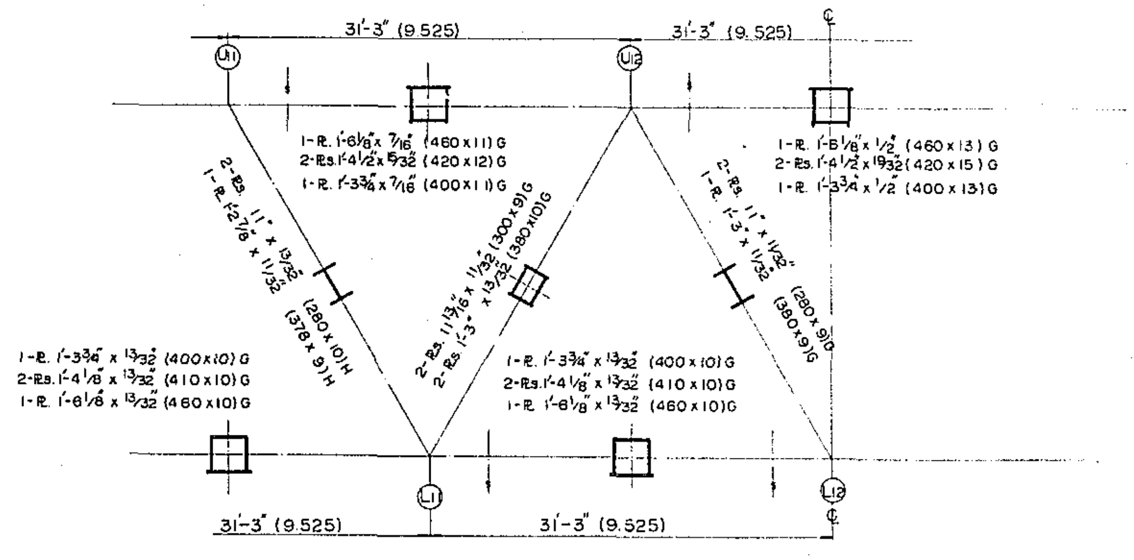
ELEVATION AND PLAN
0 10' 20' 30'

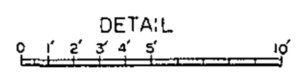
SECTION
0 10' 20'

NOTE
FIGURE IN PARENTHESES ARE SHOWN

CHITTAGONG EAST PAKISTAN
KARNAPHULI RIVER BRIDGE
GENERAL VIEW 3 SPAN CONTINUOUS TRUSS GIRDER, TYPE A,B
OCT. 1965 | II − 3





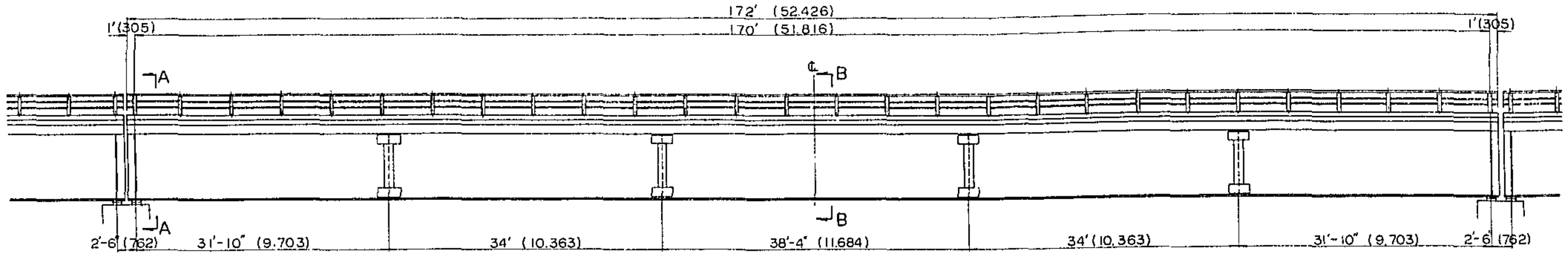


NOTES

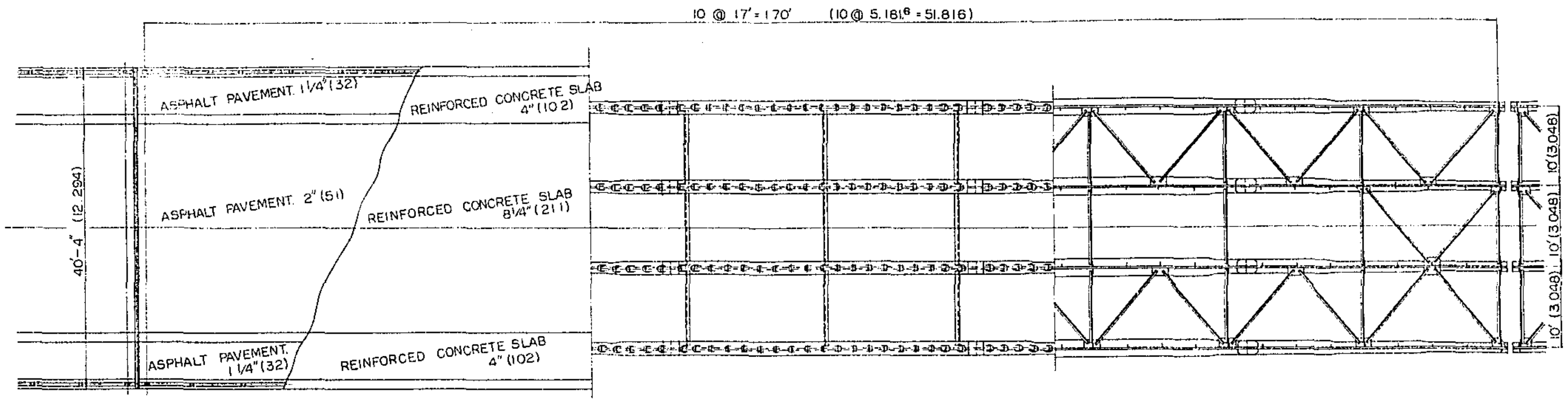
1. FIGURES IN PARENTHESES ARE SHOWN IN MM
2. "H" SHOWS HIGH-TENSILE STEEL FOR WELDED STRUCTURE (JIS G 3106 SM50A)
   "G" SHOWS ROLLED STEEL FOR GENERAL STRUCTURE (JIS G 3101 SS41)

CHITTAGONG EAST PAKISTAN KARNAPHULI RIVER BRIDGE	
TYPICAL DETAIL 3 SPAN CONTINUOUS TRUSS GIRDER	
OCT. 1965	II - 4

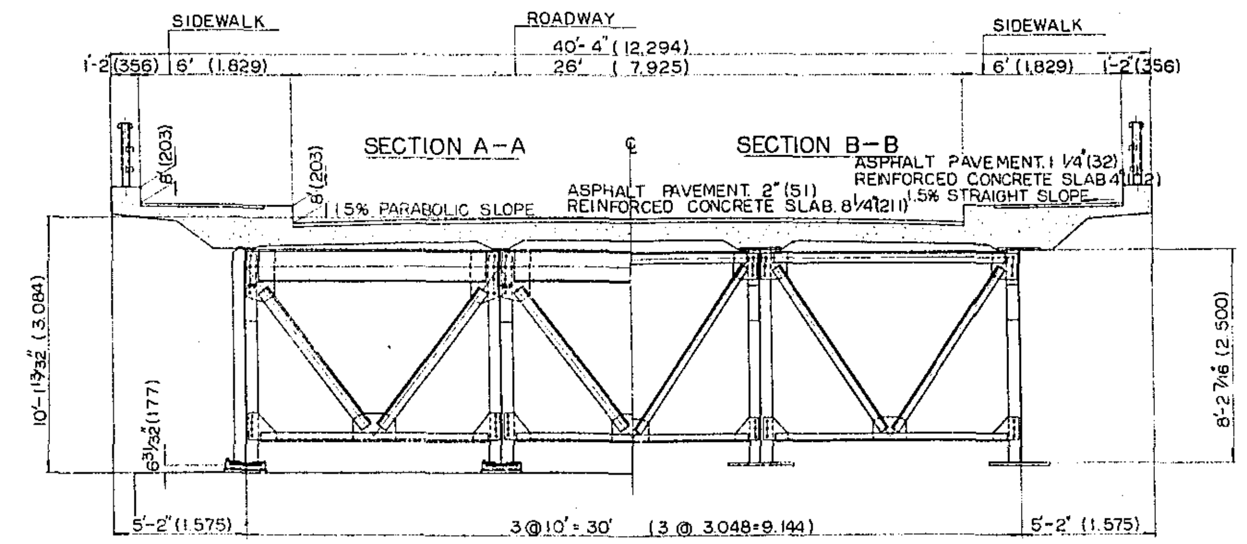
ELEVATION



PLAN



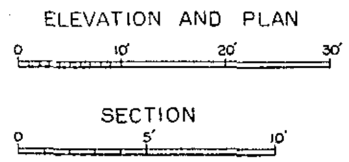
SECTION



DIMENSIONS OF MAIN GIRDERS

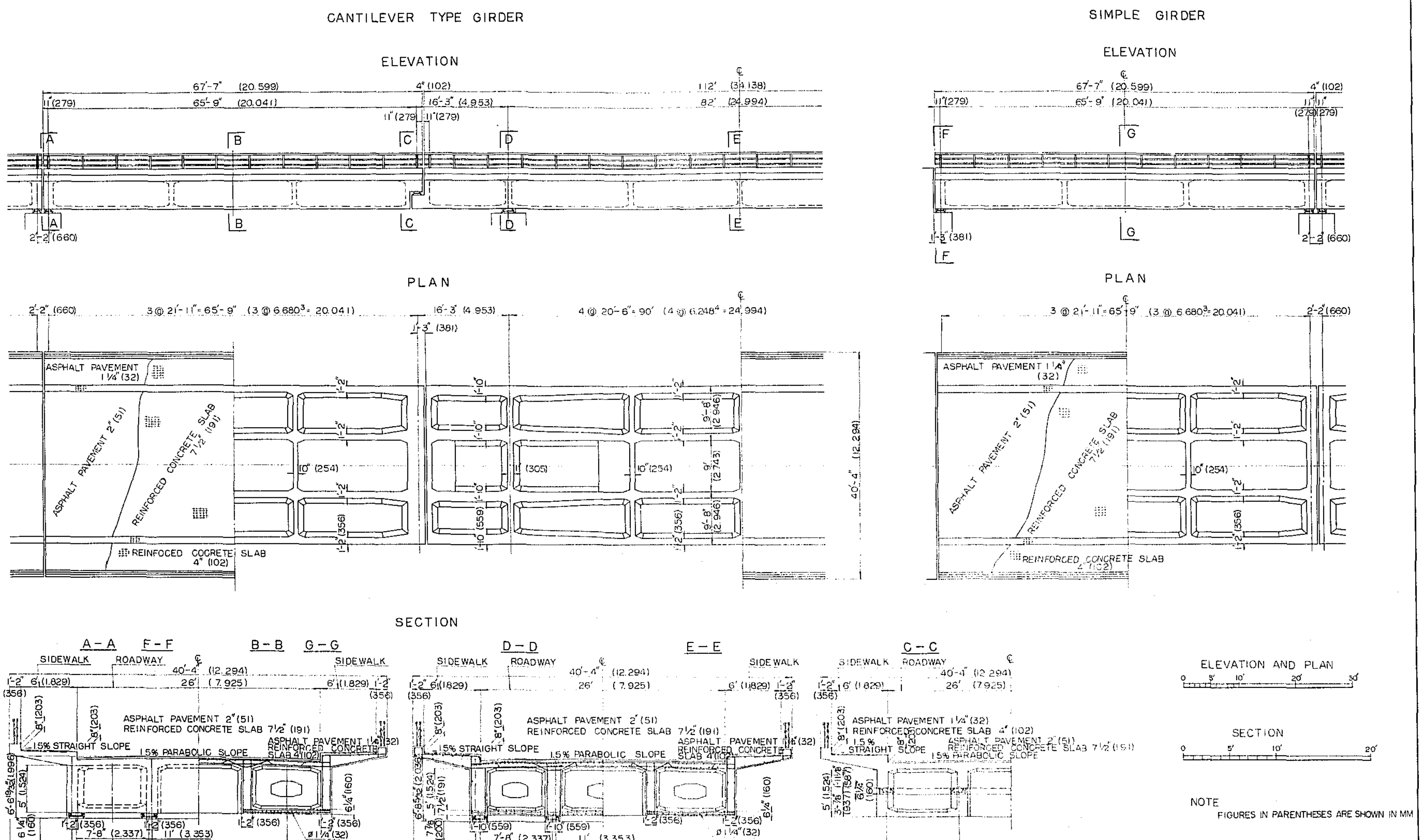
OUTSIDE GIRDER			
END.	UP. FLANGE PL	2 @ 7 7/8" x 13/32" x 11' (2 @ 200 x 10 x 3.353)	G
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 10'-8" (2 @ 2500 x 10 x 3.251)	G
	LOW. FLANGE PL	2 @ 11 13/16" x 5/8" x 10'-4" (2 @ 300 x 16 x 3.150)	G
2ND.	UP. FLANGE PL	2 @ 9 7/8" x 9/16" x 11'-6" (2 @ 250 x 14 x 3.505)	H
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 11'-10" (2 @ 2500 x 10 x 3.607)	H
	LOW. FLANGE PL	2 @ 1'-4 1/8" x 7/8" x 11'-10" (2 @ 410 x 22 x 3.607)	H
3RD.	UP. FLANGE PL	2 @ 1'-2 9/16" x 7/8" x 15' (2 @ 370 x 22 x 4.572)	H
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 15' (2 @ 2500 x 10 x 4.572)	H
	LOW. FLANGE PL	2 @ 1'-8 1/2" x 1 1/4" x 15' (2 @ 520 x 32 x 4.572)	H
4TH.	UP. FLANGE PL	2 @ 1'-6 1/8" x 1 3/32" x 21'-6" (2 @ 460 x 28 x 6.553)	H
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 21'-6" (2 @ 2500 x 10 x 6.553)	H
	LOW. FLANGE PL	2 @ 2'-1 3/16" x 1 7/16" x 21'-6" (2 @ 640 x 36 x 6.553)	H
CENTER	UP. FLANGE PL	1 @ 1'-8 7/8" x 1 3/32" x 54' (1 @ 530 x 28 x 16.459)	H
	WEB. PL	1 @ 8'-2 7/16" x 13/32" x 54' (1 @ 2500 x 10 x 16459)	H
	LOW. FLANGE PL	1 @ 2-4 3/4" x 1 7/16" x 54'-8" (1 @ 730 x 36 x 16.662)	H

INSIDE GIRDER			
END.	UP. FLANGE PL	2 @ 7 7/8" x 13/32" x 11' (2 @ 200 x 10 x 3.353)	G
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 10'-8" (2 @ 2500 x 10 x 3.251)	G
	LOW. FLANGE PL	2 @ 11 13/16" x 5/8" x 10'-4" (2 @ 300 x 16 x 3.150)	G
2ND.	UP. FLANGE PL	2 @ 9 7/16" x 9/16" x 11'-6" (2 @ 240 x 14 x 3.505)	H
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 11'-10" (2 @ 2.500 x 10 x 3.607)	H
	LOW. FLANGE PL	2 @ 1'-4 15/16" x 7/8" x 11'-10" (2 @ 430 x 22 x 3.607)	H
3RD.	UP. FLANGE PL	2 @ 1'-1 3/8" x 7/8" x 15' (2 @ 340 x 22 x 4.572)	H
	WEB. PL	2 @ 8'-2 7/16" x 13/32" x 15' (2 @ 2.500 x 10 x 4.572)	H
	LOW. FLANGE PL	2 @ 1'-9 1/4" x 1 1/4" x 15' (2 @ 540 x 32 x 4.572)	H
4TH.	UP. FLANGE PL	2 @ 1'-4 15/16" x 1 3/32" x 21'-6" (2 @ 430 x 28 x 6.553)	H
	WEB PL	2 @ 8'-2 7/16" x 13/32" x 21'-6" (2 @ 2500 x 10 x 6.553)	H
	LOW. FLANGE PL	2 @ 2'-2 3/8" x 1 7/16" x 21'-6" (2 @ 670 x 36 x 6.553)	H
CENTER	UP. FLANGE PL	1 @ 1'-7 11/16" x 1 3/32" x 54' (1 @ 500 x 28 x 16.459)	H
	WEB. PL	1 @ 8'-2 7/16" x 13/32" x 54' (1 @ 2.500 x 10 x 16.459)	H
	LOW. FLANGE PL	1 @ 2'-6 5/16" x 1 7/16" x 54'-8 (1 @ 770 x 36 x 16.662)	H



NOTE
1. FIGURES IN PARENTHESES ARE SHOWN IN MM
2. IN THE TABLE SECTIONS OF MAIN GIRDERS
"H" SHOWS HIGH-TENSILE STEEL FOR WELDED STRUCTURE (JIS G 3106 SM50AB)
"G" SHOWS ROLLED STEEL FOR GENERAL STRUCTURE (JIS G 3101 SS41)

CHITTAGONG EAST PAKISTAN KARNAPHULI RIVER BRIDGE	
TYPICAL DETAIL SIMPLE COMPOSITE GIRDER, TYPE A	
OCT. 1965	II - 5



CHITTAGONG EAST PAKISTAN

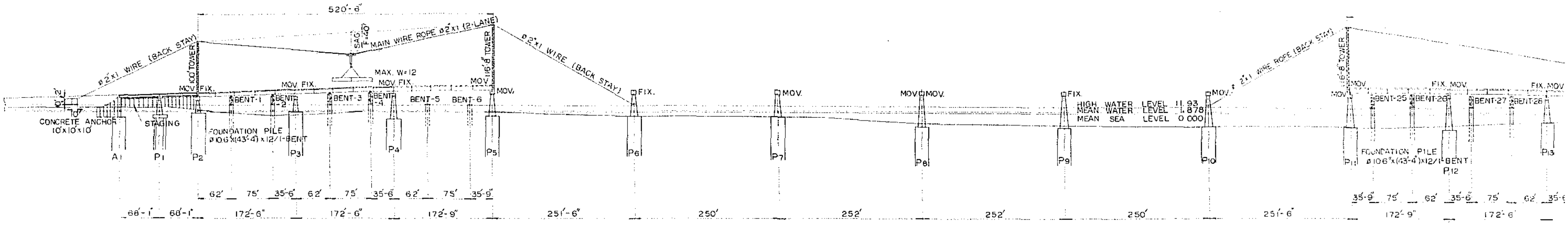
KARNAPHULI RIVER BRIDGE

TYPICAL DETAIL
CONCRETE GIRDERS, TYPE A

OCT. 1965 | II – 6

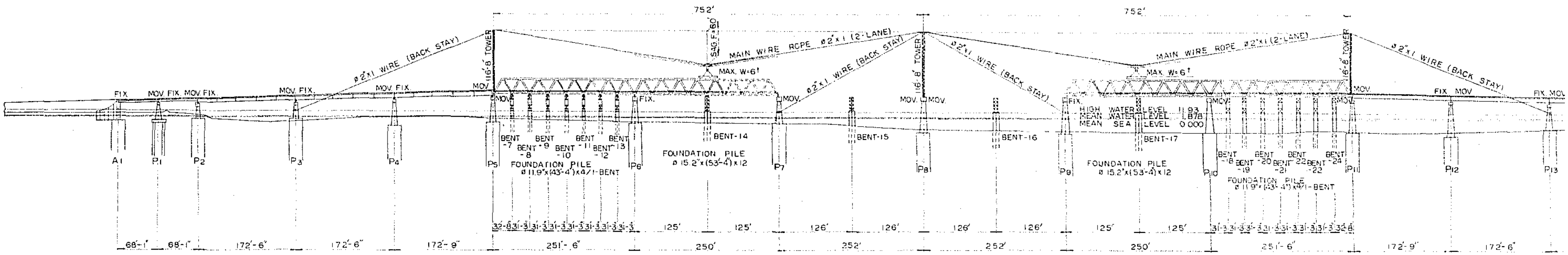
CONCRETE GIRDER AND COMPOSITE GIRDER

ELEVATION (1)

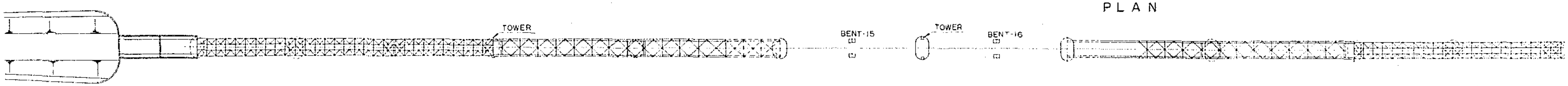


TRUSS GIRDER

ELEVATION (2)

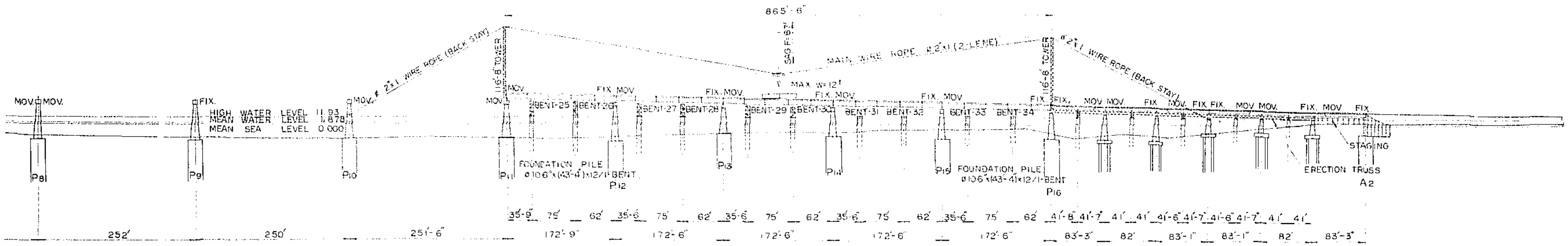


PLAN



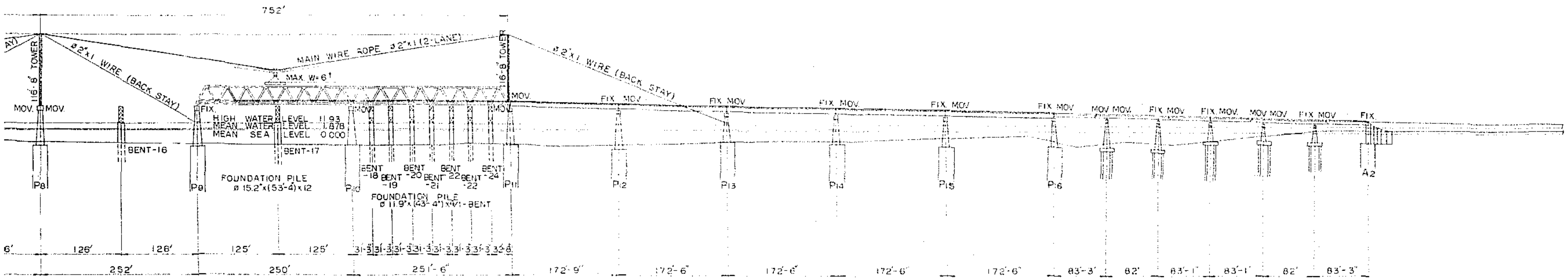
CONCRETE GIRDER AND COMPOSITE GIRDER

ELEVATION (1)



TRUSS GIRDER

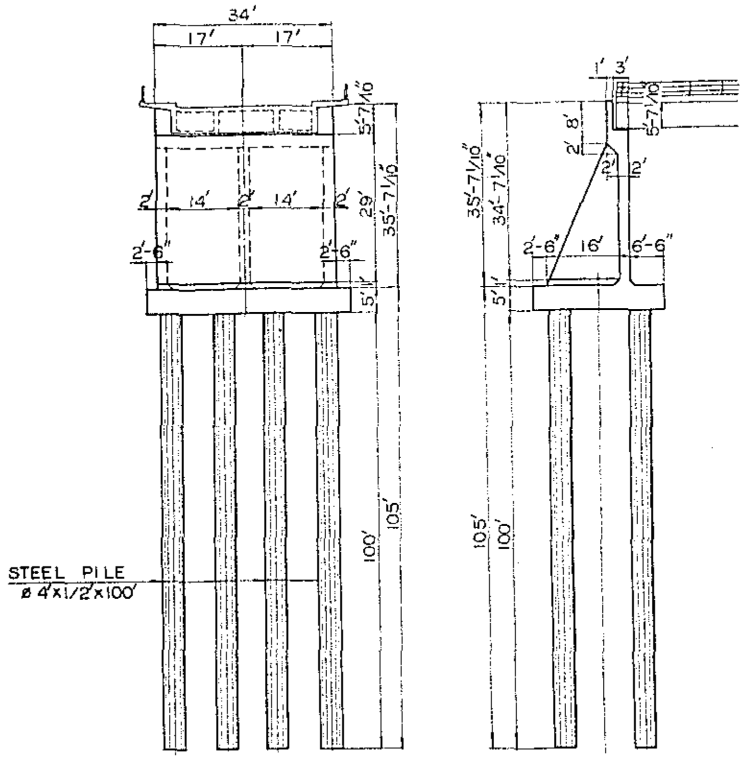
ELEVATION (2)

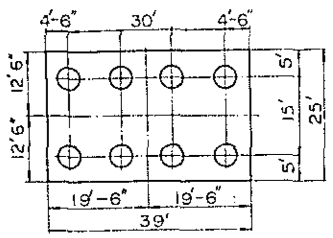


PLAN

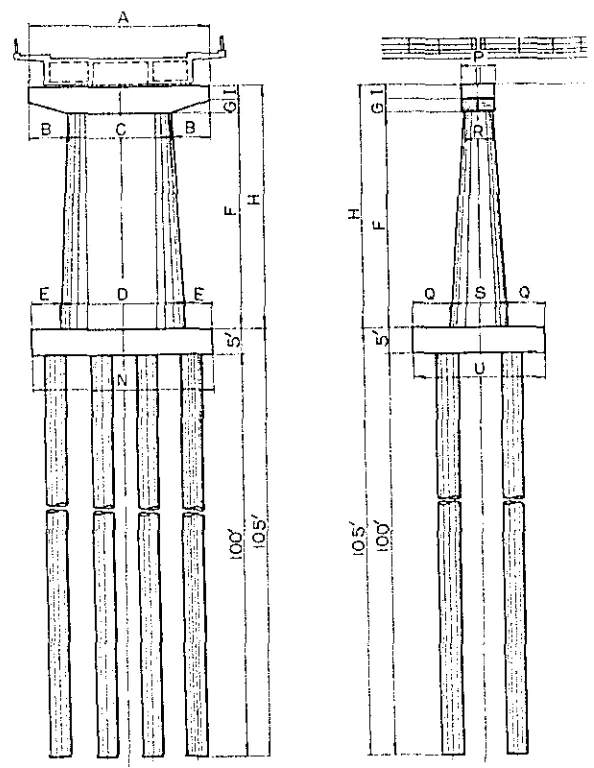
CHITTAGONG EAST PAKISTAN KARNARHULI RIVER BRIDGE	
GENERAL ARRANGEMENT ERECTION WORKS TYPE A	
OCT. 1965	II – 7

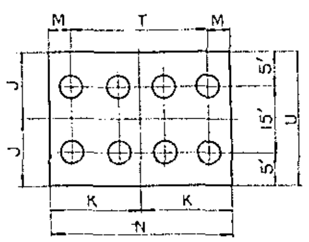
ABUTMENT 1. 2



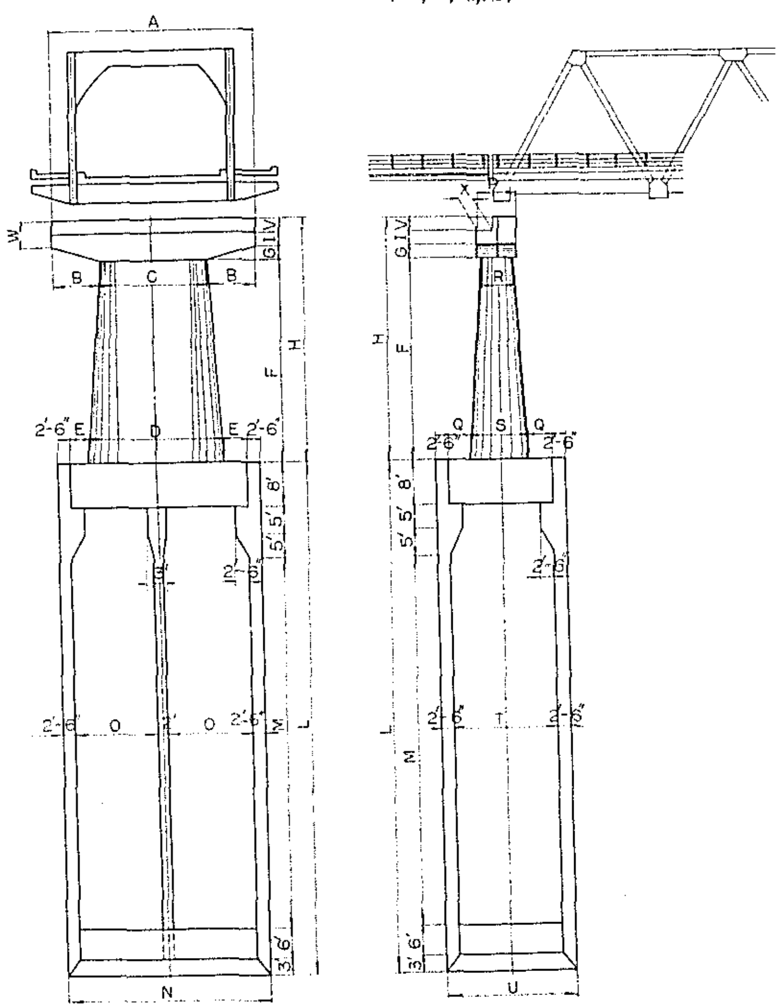


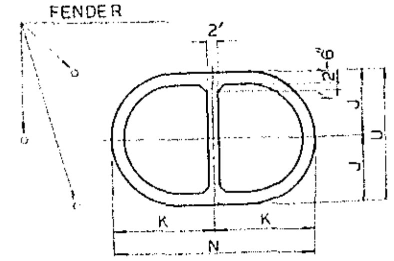
PIER (P2~P3, P5~P10, P12~P15, P17~P21)





PIER (P1, P4, P11, P16)





SECTION

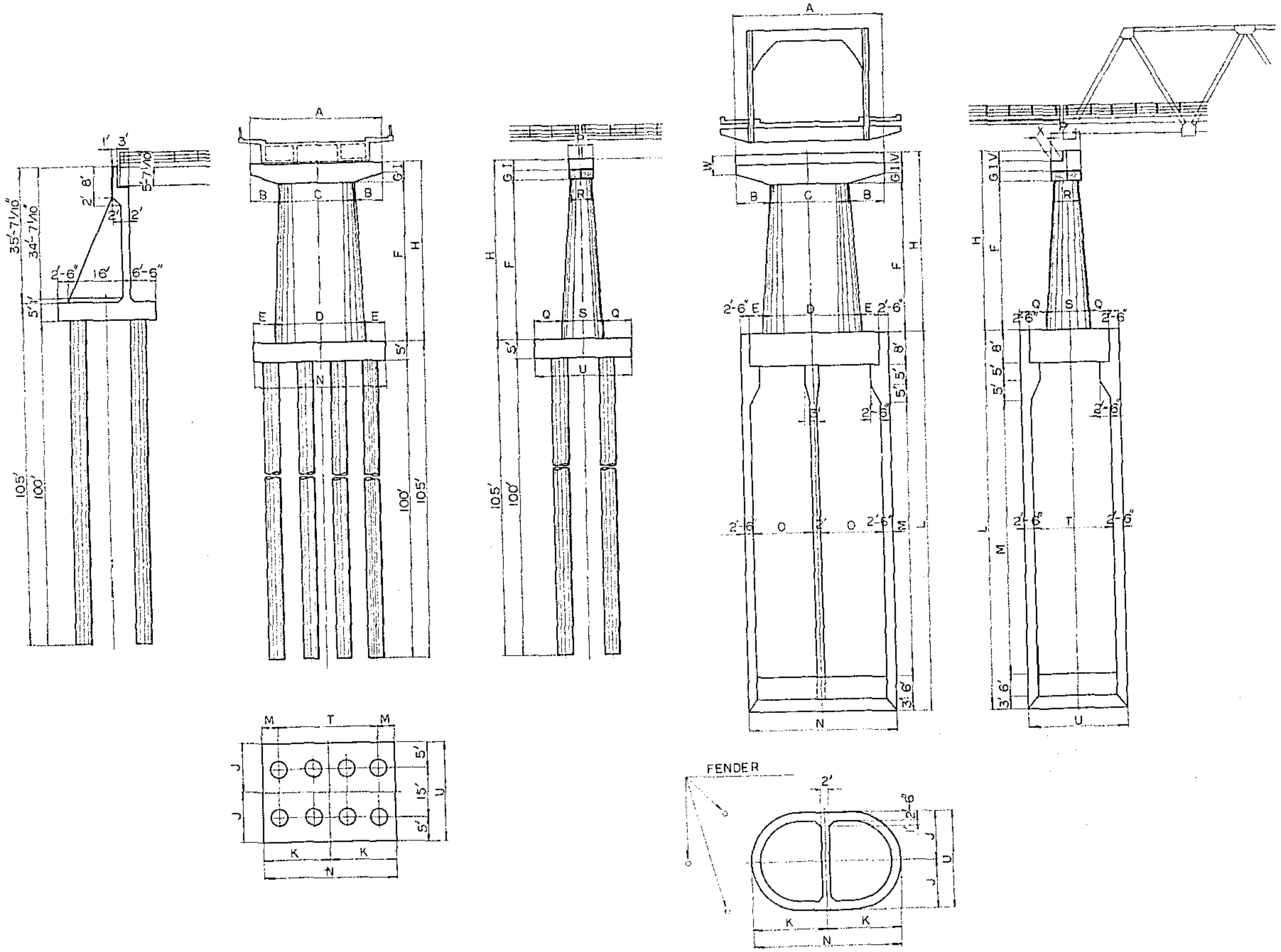
0 10' 20' 40'

DEMENSIONS OF

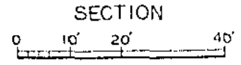
	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	
A	34'	38'	38'	38'	38'	39'	39'	39'	39'	39'	38'	38'	38'	3
B	7'-6"	9'	9'	9'	9'	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'	9'	9'	9
C	19'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	2
D	21'-5"	22'-5"	22'-5"	24'-5"	24'-5"	24'-5"	24'-5"	25'-6"	25'-6"	25'-6"	25'-6"	25'-6"	24'-5"	24'
E	5'-9 1/2"	5'-3 1/2"	5'-3 1/2"	2'-3 1/2"	2'-3 1/2"	2'-3 1/2"	2'-3 1/2"	4'-3"	4'-3"	4'-3"	4'-3"	4'-3"	2'-3 1/2"	2'-3
F	25'	24'-6"	24'	41'	41'-4"	39'	39'	55'	55'	55'	55'	55'	41'	4
G	2'-6"	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3
H	30'	30'	30'	47'	47'	47'	47'	63'	63'	63'	63'	63'	47'	4
I	2'-6"	2'-6"	3'	3'	2'-8"	5'	5'	5'	5'	5'	2'-8"	3'	3'	3
J	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-
K	19'	19'	19'	17'	17'	17'	17'	19'-6"	19'-6"	19'-6"	19'-6"	19'-6"	17'	1
L		103'	104'-6"	92'-6"	131'	131'	130'	115'	114'	115'	115'	76'-6"	90'-6"	8
M	4'	76'	77'-6"	65'-6"	104'	104'	103'	88'	87'	88'	88'	49'-6"	63'-6"	6
N	38'	38'	38'	34'	34'	34'	34'	39'	39'	39'	39'	39'	34'	3
O		15'-6"	15'-6"	13'-6"	13'-6"	13'-6"	13'-6"	16'	16'	16'	16'	16'	13'-6"	13'-
P	6'-4"	6'-4"	6'-4"	6'-4"	7'-4"	9'	9'	9'	9'	9'	7'-4"	6'-4"	6'-4"	6'-
Q	8'-1/2"	5'-6 1/2"	5'-6 1/2"	4'-9"	4'-9"	4'-3"	4'-3"	6'-9"	6'-9"	6'-9"	4'-3"	4'-3"	4'-9"	4'-
R	6'	6'	6'	6'	6'	7'	7'	7'	7'	7'	6'	6'	6'	6
S	8'-5"	8'-5"	8'-5"	10'-6"	10'-6"	11'-6"	11'-6"	12'-6"	12'-6"	12'-6"	11'-6"	11'-6"	11'-6"	10'-
T	30'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	2
U	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25
V	3'			2'-2"							2'-2"			
W	5'-6"			5'-8"							5'-8"			
X	3'			2'-11"							2'-11"			

KA

PIER (P2~P3, P5~P10, P12~P15, P17~P21) PIER (P1, P4, P11, P16)



SECTION



DEMENSIONS OF

	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13	P14	P15	P16	P17	P18	P19	P20	P21
A	34'	38'	38'	38'	38'	39'	39'	39'	39'	39'	38'	38'	38'	38'	38'	38'	34'	34'	34'	34'	34'
B	7'-6"	9'	9'	9'	9'	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'	9'	9'	9'	9'	9'	7'-6"	7'-6"	7'-6"	7'-6"	7'-6"
C	19'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	19'	19'	19'	19'	19'
D	21'-5"	22'-5"	22'-5"	24'-5"	24'-5"	24'-5"	24'-5"	25'-6"	25'-6"	25'-6"	25'-6"	25'-6"	24'-5"	24'-5"	24'-5"	24'-5"	23'-2"	23'-2"	21'-5"	21'-5"	21'-5"
E	5'-9½"	5'-3½"	5'-3½"	2'-3½"	2'-3½"	2'-3½"	2'-3½"	4'-3"	4'-3"	4'-3"	4'-3"	4'-3"	2'-3½"	2'-3½"	2'-3½"	2'-3½"	2'-11"	2'-11"	5'-9½"	5'-9½"	5'-9½"
F	25'	24'-6"	24'	41'	41'-4"	39'	39'	55'	55'	55'	55'	55'	41'	41'	41'	41'-6"	42'	42'	24'-6"	24'-6"	24'-6"
G	2'-6"	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	2'-6"	2'-6"	2'-6"	2'-6"	2'-6"
H	30'	30'	30'	47'	47'	47'	47'	63'	63'	63'	63'	63'	47'	47'	47'	47'	47'	47'	30'	30'	30'
I	2'-6"	2'-6"	3'	3'	2'-8"	5'	5'	5'	5'	5'	2'-8"	3'	3'	3'	3'	2'-6"	2'-6"	2'-6"	2'-6"	2'-6"	2'-6"
J	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"
K	19'	19'	19'	17'	17'	17'	17'	19'-6"	19'-6"	19'-6"	19'-6"	19'-6"	17'	17'	17'	17'	17'	17'	19'	19'	19'
L		103'	104'-6"	92'-6"	131'	131'	130'	115'	114'	115'	115'	76'-6"	90'-6"	87'	82'-6"	83'					
M	4'	76'	77'-6"	65'-6"	104'	104'	103'	88'	87'	88'	88'	49'-6"	63'-6"	60'	55'-6"	56'	4'	4'	4'	4'	4'
N	38'	38'	38'	34'	34'	34'	34'	39'	39'	39'	39'	39'	34'	34'	34'	34'	38'	38'	38'	38'	38'
O		15'-6"	15'-6"	13'-6"	13'-6"	13'-6"	13'-6"	16'	16'	16'	16'	16'	13'-6"	13'-6"	13'-6"	13'-6"					
P	6'-4"	6'-4"	6'-4"	6'-4"	7'-4"	9'	9'	9'	9'	9'	7'-4"	6'-4"	6'-4"	6'-4"	6'-4"	6'-4"	6'-4"	6'-4"	6'-4"	6'-4"	6'-4"
Q	8'-½"	5'-6½"	5'-6½"	4'-9"	4'-9"	4'-3"	4'-3"	6'-9"	6'-9"	6'-9"	4'-3"	4'-3"	4'-9"	4'-9"	4'-9"	4'-9"	7'-3"	7'-3"	8'-½"	8'-½"	8'-½"
R	6'	6'	6'	6'	6'	7'	7'	7'	7'	7'	6'	6'	6'	6'	6'	6'	6'	6'	6'	6'	6'
S	8'-5"	8'-5"	8'-5"	10'-6"	10'-6"	11'-6"	11'-6"	12'-6"	12'-6"	12'-6"	11'-6"	11'-6"	11'-6"	10'-6"	10'-6"	10'-6"	10'-6"	10'-6"	8'-5"	8'-5"	8'-5"
T	30'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	30'	30'	30'	30'	30'
U	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'
V	3'			2'-2"							2'-2"					2'-2"					
W	5'-6"			5'-8"							5'-8"					5'-6"					
X	3'			2'-11"							2'-11"					3'					

CHITTAGONG EAST PAKISTAN

KARNAPHULI RIVER BRIDGE

TYPICAL DETAIL
SUBSTRUCTURES, TYPE A

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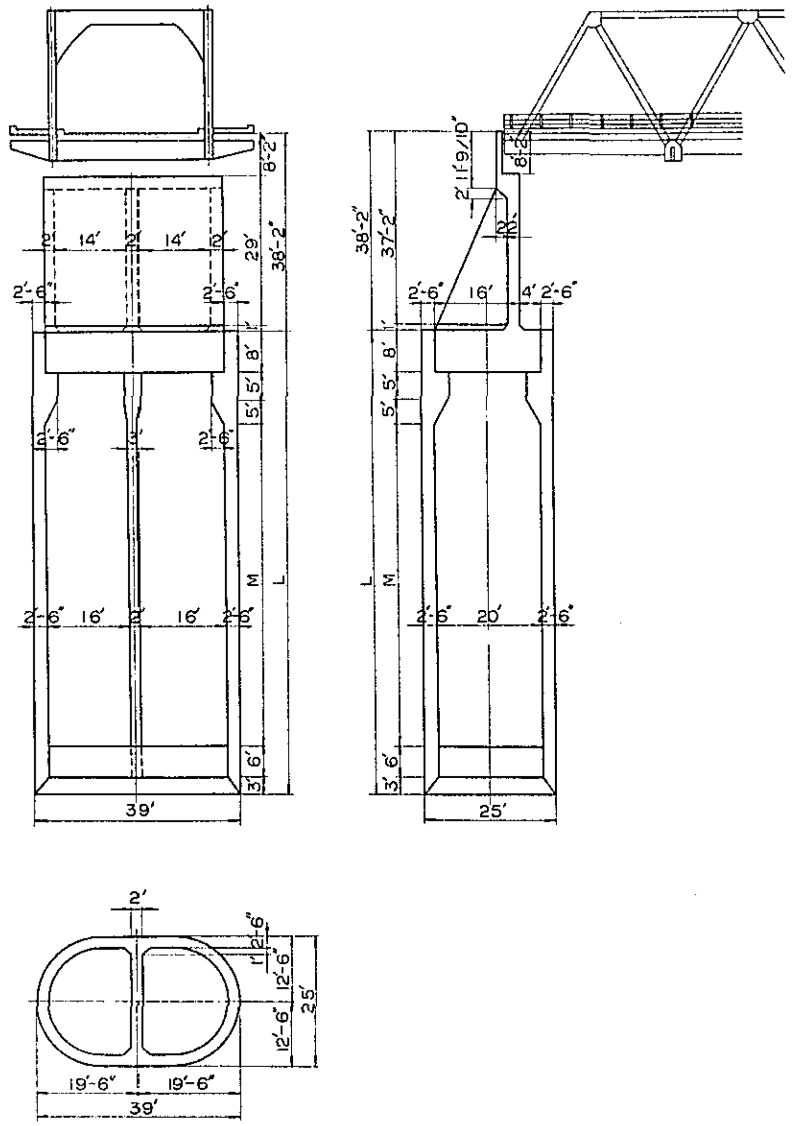
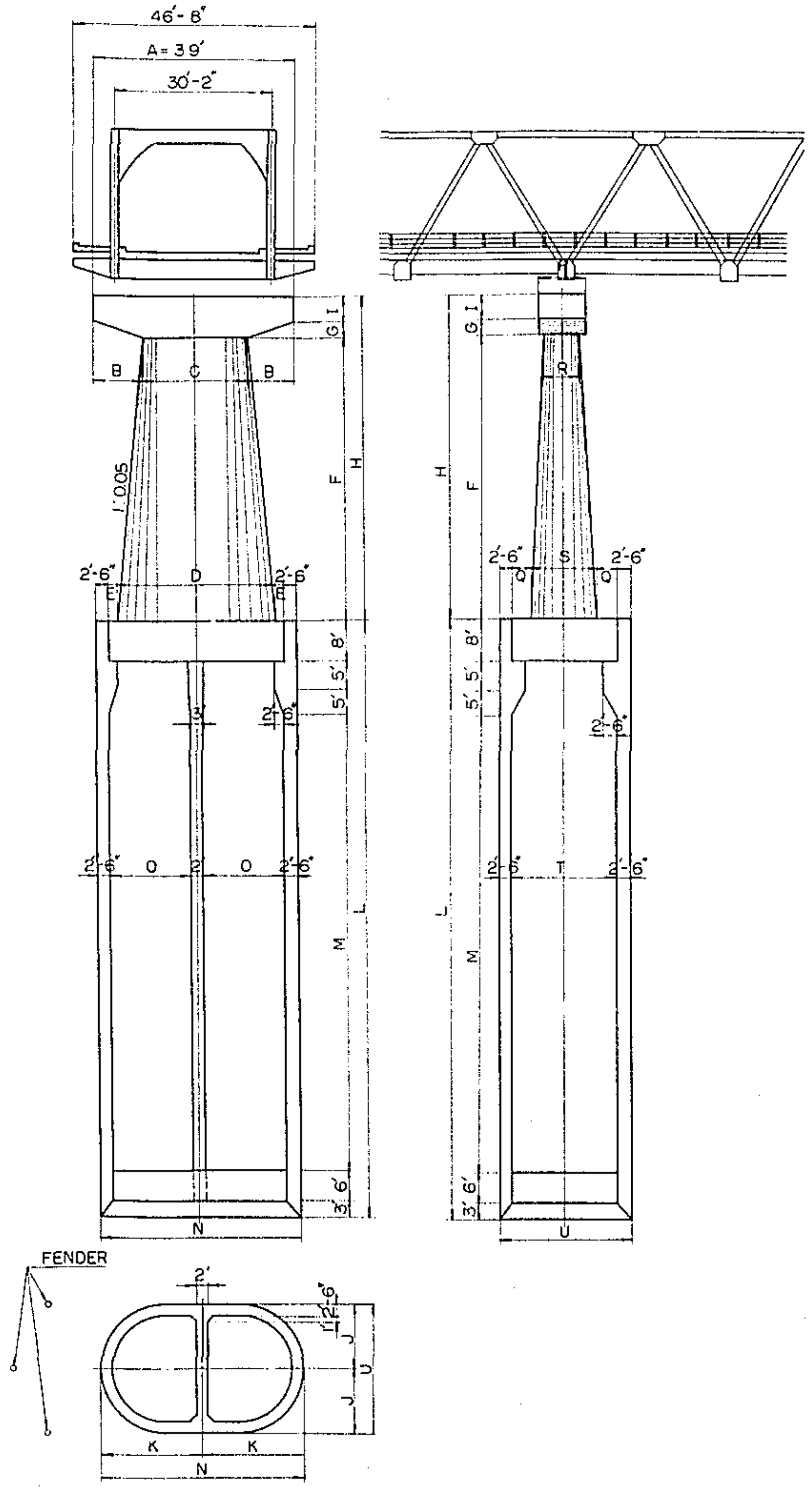
PIER

ABUTMENT





DEMENSIONS OF

	P1	P2	P3	P4	P5	P6	P7	P8	P9	P10	P11	P12	P13
A	39'	39'	39'	39'	39'	39'	39'	39'	39'	39'	39'	39'	39'
B	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"	9'-6"
C	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'
D	22'-2"	22'-2"	23'-10"	23'-10"	23'-10"	25'-6"	25'-6"	25'-6"	25'-6"	25'-6"	25'-6"	22'-2"	22'-2"
E	10'-10"	10'-10"	5'-2"	5'-2"	5'-2"	8'-6"	8'-6"	8'-6"	8'-6"	3'-6"	3'-6"	7'-10"	10'-10"
F	22'	22'	39'	39'	39'	55'	55'	55'	55'	55'	55'	22'	22'
G	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'	3'
H	30'	30'	47'	47'	47'	63'	63'	63'	63'	47'	47'	30'	30'
I	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'	5'
J	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"
K	19'	19'	17'	17'	17'	19'-6"	19'-6"	19'-6"	19'-6"	17'	17'	17'	19'
L	104'-6"	104'-6"	131'	131'	130'	115'	114'	115'	115'	90'-6"	87'	83'	94'-6"
M	77'-6"	77'-6"	104'	104'	103'	88'	87'	88'	88'	63'-6"	60'	56'	67'-6"
N	38'	38'	34'	34'	34'	39'	39'	39'	39'	34'	34'	34'	38'
O	15'-6"	15'-6"	13'-6"	13'-6"	13'-6"	16'	16'	16'	16'	13'-6"	13'-6"	13'-6"	15'-6"
P	9'	9'	9'	9'	9'	9'	9'	9'	9'	9'	9'	9'	9'
Q	10'-10"	10'-10"	9'-2"	9'-2"	9'-2"	7'-6"	7'-6"	7'-6"	7'-6"	7'-6"	7'-6"	10'-10"	10'-10"
R	7'	7'	7'	7'	7'	7'	7'	7'	7'	7'	7'	7'	7'
S	9'-2"	9'-2"	10'-10"	10'-10"	10'-10"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	12'-6"	9'-2"	9'-2"
T	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'	20'
U	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'	25'

SECTION

0 10' 20' 30' 40'

CHITTAGONG EAST PAKISTAN

KARNAPHULI RIVER BRIDGE

TYPICAL DETAIL
SUBSTRUCTURES , TYPE B

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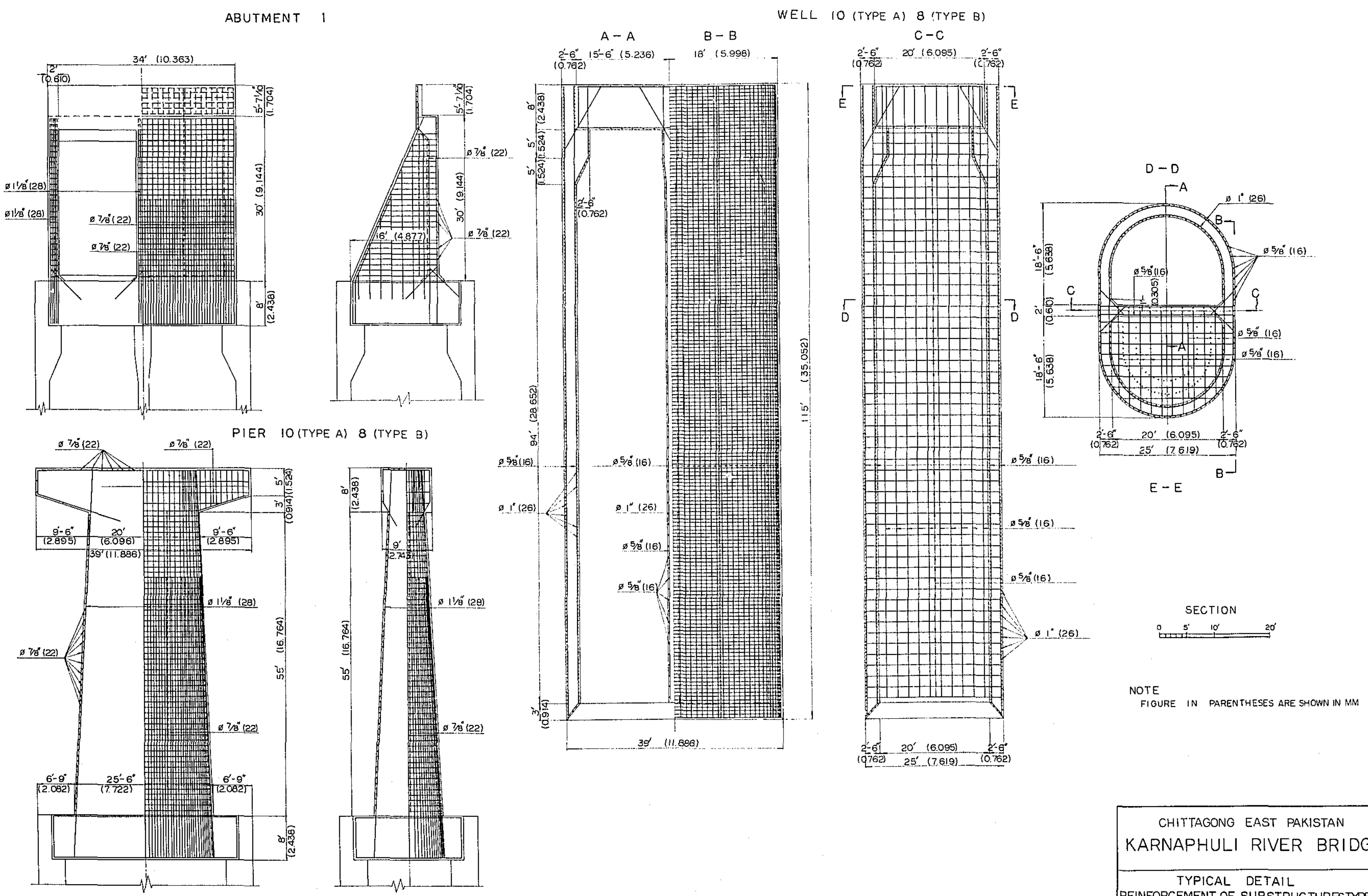

NOTE
FIGURE IN PARENTHESES ARE SHOWN IN MM

CHITTAGONG EAST PAKISTAN

KARNAPHULI RIVER BRIDGE

TYPICAL DETAIL

REINFORCEMENT OF SUBSTRUCTURES TYPE A,B

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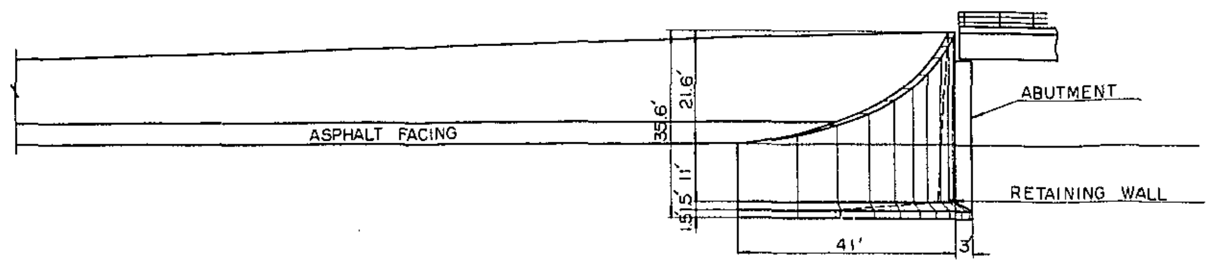
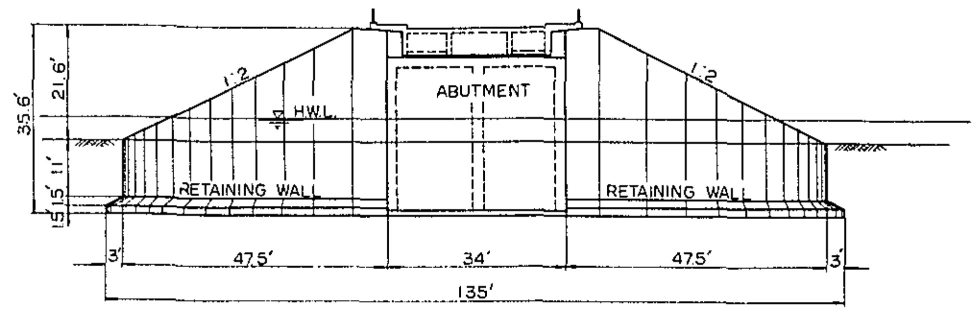
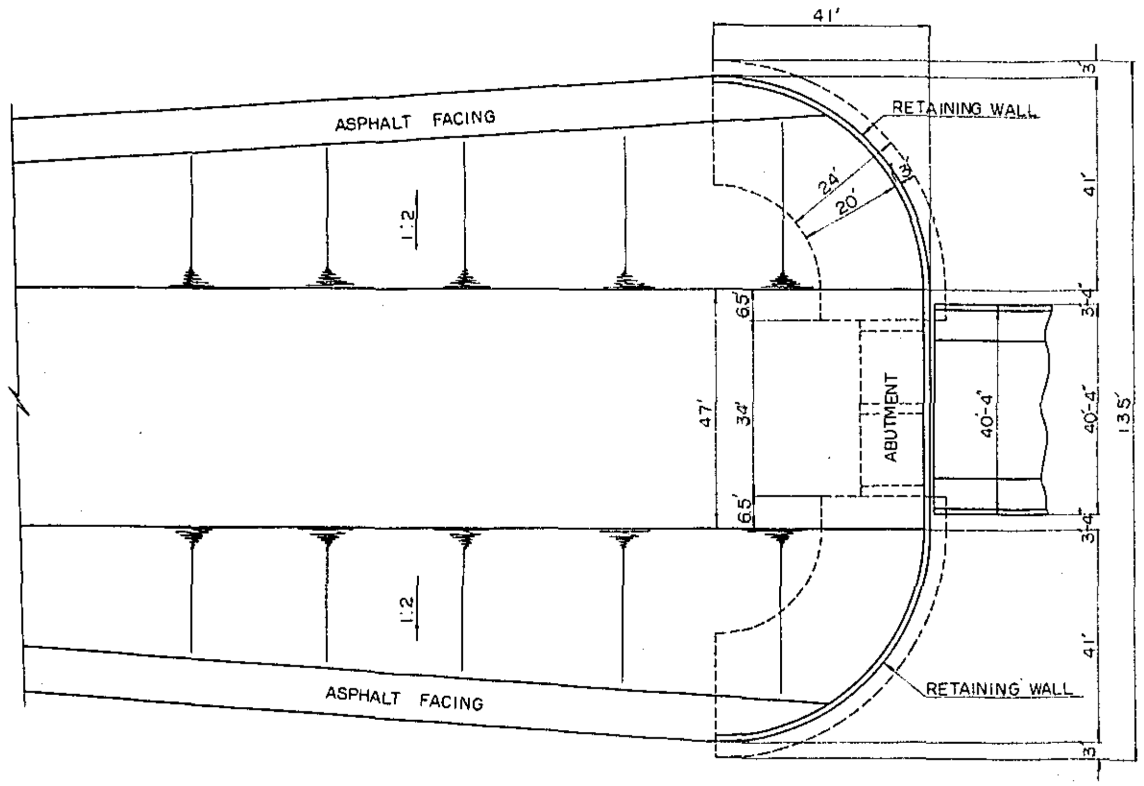
SIDE VIEW

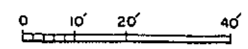


FRONT VIEW



PLAN





CHITTAGONG EAST PAKISTAN KARNAPHULI RIVER BRIDGE	
APPROACH ROAD	
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