# CHAPTER VI
# MINIMUM RIVER WIDTH

## 1. Lacey's Formula.

According to G. Lacey's study (114 GB) made on the rivers and canals in India and Pakistan, a minimum stable river width of large rivers in an alluvial plain is expressed by the following formula.

$$W_{min} = 2.67 \sqrt{Q} \qquad (1)$$

where

$W_{min}$ = Minimum width in stable channel (ft)
$Q$ = Discharge (cfs).

The distribution of design discharge (100-year flood) has already been mentioned in Chapter IV, which gives about 90,000 $m^3/s$ as the average value of mean velocity on the stretch from Bahadurabad to the confluence with the Ganges. If the above formula is applicable also to the Jamuna River, the minimum stable river width of this river will be

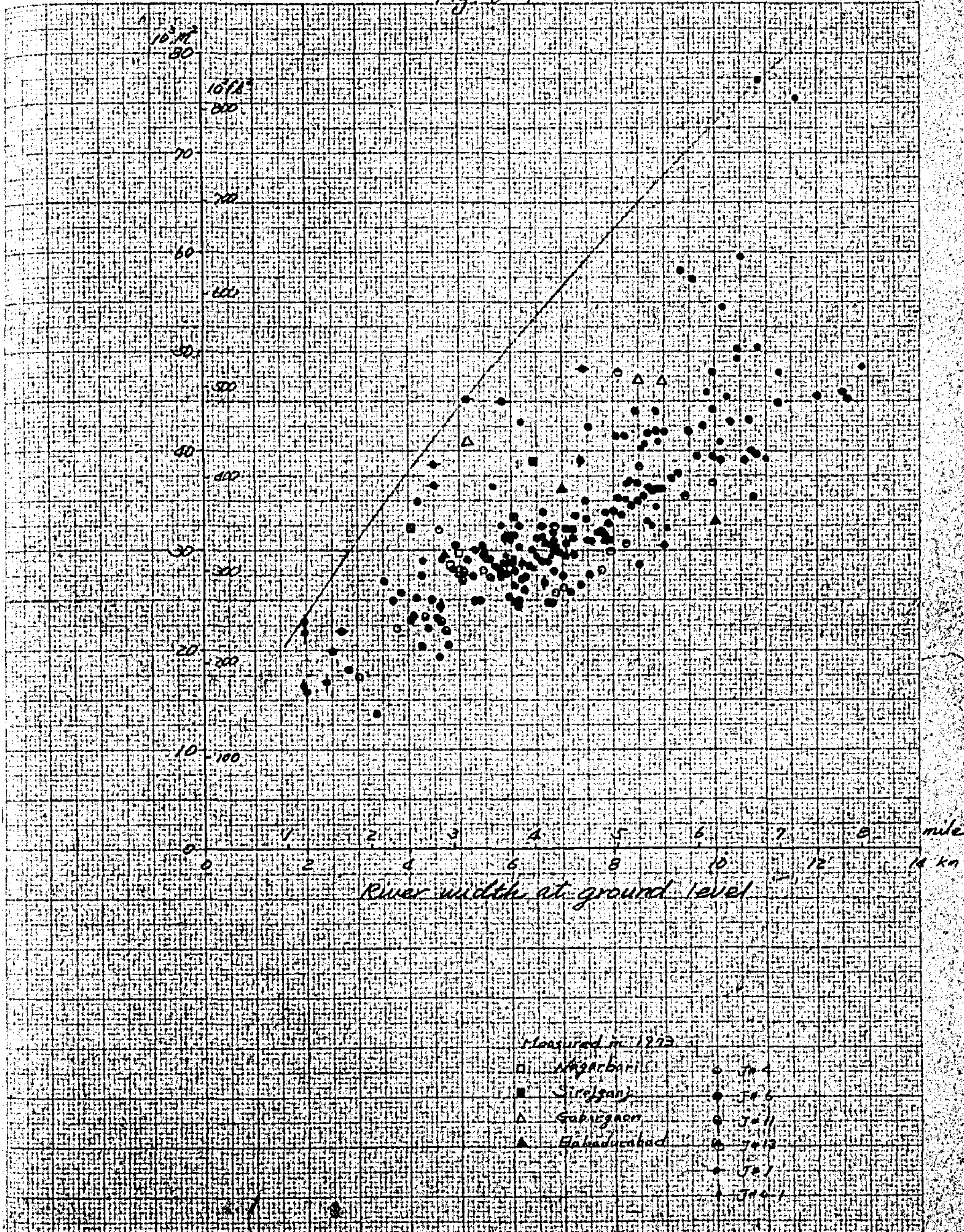
$$W_{min} = 4,900 \text{ ft} = 1,500 \text{ m}.$$

## 2. Nature of the Present Jamuna River.

Relationship between river width at the ground level and the corresponding water area below the ground level was studied making use of the results of cross surveying made by BWDB over the stretch of about 220 km from the confluence with the Ganges to the upstream of Bahadurabad every year from the dry season of 1964/65 to the dry season of 1971/72, the result of which is shown in Fig. 6-1. This figure indicates that there exists a minimum river width for a specified water area.

According to calculations, the water stage will be raised 3 to 5 m above the ground level in case the maximum flood discharge in 1970, 2,700,000 cfs (76,460 $m^3/s$), flows, under $n = 0.02$, through the cross sections surveyed in the 1969/70 dry season. Similarly, the water stages will be raised 3 to 5 m above the ground level both in case the design discharge (100-year flood) flows through the average cross sections of those surveyed during the period from 1964 to 1971 and in case the 100-year flood flows through the cross sections surveyed in 1973. Mean velocity as the average on a water area, $v_M$, was then computed at about 1.6 m/s (5.2 ft/s) on the average over the whole stretch of the river in all the three cases.

Fig. 6-1



$10^3 m^2$
$10^3 ft^2$
80
800
70
700
60
600
50
500
40
400
30
300
20
200
10
100
0
1
2
3
4
5
6
7
8
mile
0
2
4
6
8
10
12
14 km
River width at ground level
Measured in 1973
Nagarbari
Sirajganj
Gabargaon
Bahadurabad
J-4
J-6
J-11
J-13
J-1
J-1

We assume that flood flow is confined within two embankments on the both banks and the water level is raised by about 4 m above the ground level according to the calculation in case a discharge nearly equal to the design discharge namely 90,000 $m^3/s$ flows through the river channel.

Now, let Q be the discharge 90,000 $m^3/s$, the discharge may be expressed by

$$Q = A_1 v_{m1} + A_2 v_{m2} \qquad (a)$$

where $A_1$ and $v_{m1}$ are water area above the ground level and mean velocity of the water area, and $A_2$ and $v_{m2}$ are respective values concerning below the ground level. Giving the value of discharge Q, we are going to obtain the value of $A_2$ using the equation (a).

Since the equation (a) is nonlinear, we consider to make some approximation with the view of obtaining an approximate value of $A_2$. First, we rewrite the equation (a) as follows.

$$A_2 = Q/v_{m2} - A_1(v_{m1}/v_{m2}) \qquad (b)$$

In this equation, if we make an approximation that $v_S/v_m$ is used in place of $v_{m1}/v_{m2}$ and $v_m$ in place of $v_{m2}$, we can obtain the value of $A_2$ when the value of $A_1$ is given.

Now, we assume that $v_{m2}$ is about 1.6 m/s and the rise of the water level is about 4 m in case of the discharge 90,000 $m^3/s$ refering to the preliminary calculations previously mentiond. If we thus assume a value of river width between a pair of guide banks, we will get a value of $A_1$ and we will obtain a value for $A_2$ from the equation (b). We can read on Fig.6-1 a minimum river width for this value. By iterating this procedure until the read value becomes equal to the assumed value, we can estimate an approximate value of the minimum river width corresponding to the discharge 90,000 $m^3/s$ (3,180,000 c.f.s.).

Relationship between the mean velocity $v_m$ and the surface velocity $v_s$ on a vertical of a cross section was, in the case of the Jamuna River,

$$v_m = 0.78\ v_s$$

and, in the case of the Ganges River,

$$v_m = 0.83\ v_s$$

as already mentioned in Section 2, Chapter V. On the average, we obtain

$$v_m = 0.8\ v_s. \qquad (2)$$

Since the velocity on a water area above ground level is nearly equal to the surface velocity and the velocity below the ground level is regarded to equal to the mean velocity of the cross section, if we use the relation (2), the mean surface velocity $v_s$ for a discharge 90,000 $m^3/s$ is computed at

$$v_S = \frac{v_M\ v_s}{v_m} = 1.6/0.8 = 2\ m/s.\ (6.6\ ft/s)$$

and also, the water stage at this time is, on the average, 4 m above the ground level.

Now, if we assume a river width and devide the discharge 90,000 $m^3/s$ into two parts above and below the ground level, we can read on the Fig. 6-1 the minimum river width for the latter. By iterating this procedure untill the read value comes to equall to the assumed value, we can estimate the minimum river width corresponding to the discharge 90,000 $m^3/s$(3,180,000 cfs). Thus we get a value for the minimum width

$$W_{min} = 3,900\ m.\ (\ 12,800\ ft\ )$$

3. Minimum River Width.

On the basis of the above mentioned, 3,900 m was taken as a standard for the minimum river width of the Jamuna River in case of a flood having a recurrence interval of 100 years.

This value largely differs from that calculated by Lacey's formula. This difference is presumed to be based upon the reason that the Lacey's formula may not be applicable to a river having no clayey banks such as the Jamuna River. C. C. Inglis expressed the width of a meandering river in a flood plain by the following formula (GB 2),

$$W = 4.88\sqrt{Q} \qquad (3)$$

where

W = River width (ft)

Q = Discharge (cfs).

According to this formula, the river width for the discharge,

$Q = 90,000\ m^3/s$ (3,180,000 cfs), is comuted at

$$W = 2,700\ m\ (8,700\ ft).$$

The design discharge at Hardinge Bridge of the Ganges River is 2,500,000 cfs (70,750 $m^3/s$) and no discharge exceeding this magnitude exists in the records ever obtained, while the Jamuna River has already e experienced discharges of this order. The mean velocity at that time is of the order of 1.5 m/s and the maximum mean velocity within a cross section $v_{m\ max}$ is of the order of 2.2 m/s (7.2 ft/s) according to Fig. 5-8 given in Section 2, Chapter V. On the other hand, according to Fig. 5-7 given in the same Section, $v_M$= 2.6 m/s for the same discharge, 2,500,000 cfs, at Hardinge Bridge. Accordingly, it is presumed that the maximum mean velocity $v_{m\ max}$ and the surface velocity $v_s$ will reach to 4.6 m/s (15 ft/s) and 5.8 m/s (19 ft/s) respectively.

Also, if a discharge of $Q = 90,000\ m^3/s$ (3,180,000 cfs) passes through the Hardinge Bridge site, it is presumed from Fig. 5-7 that the average velocity on the cross section $v_M$ will be 3 m/s and the maximum mean velocity within corss section $v_{m\ max}$ and the surface velocity $v_s$ will reach to values as large as 5.6 m/s (18.4 ft/s) and 7.0 m/s (23.0 ft/s).

On the other hand, in case of the Jamuna River, these values will remain at the values of $v_M$ = 1.6 m/s (5.2 ft/s), $v_{m\ max}$ = 2.3 m/s (7.5 ft/s), and $v_s$ = 2.9 m/s (9.5 ft/s) respectively.

This computation suggests that the river width has been narrowed too much at the Harginge Bridge site.

The above-mentioned river width 3,900 m of the Jamuna River was computed on the basis of the mean velocity $v_M$= 1.6 m/s over the whole stretch of the river, while the mean velocity reaches to 2 m/s at some places of the stretch. Now, taking 2 m/s as the mean velocity and assuming the water depth above the ground level at 4 m for $Q = 90,000\ m^3/s$, we obtained a value as another minimum river width

$$W_{min} = 2,900\ m\ \ (9,500\ ft)$$

by means of the similar procedure as mentioned previously.

It is desired from the viewpoint of the bridge-work cost that the tatal cost of river training and bridge construction works will be minimized depending upon river width to be spanned. However, when we consider of river training, especially the future maintenance of the river and the future plans which may occur dangling about the river, a river width of the order of 4,000 m (13,100 ft) is required as the minimum. However the width be narrowed, it should not be made less than about 4,000 m.

# CHAPTER VII
# GUIDE BANKS

## 1. General.

It is said, in general, that braiding of a river is associated with steeper slopes and larger sediment loads than meandering, and if the slope of a stream is excessive or the discharge is increased to a relatively large magnitude, the local rate of bank scour and deposition may be of sufficient magnitude to cause the stream to braide.

The present Jamuna River is a typically braided one. There can be found no sufficient study on why the Jamuna is such a braided river. However, the major causes for braiding of this river may be sought in the matters that (1) the discharge of the Jamuna was suddenly increased by the change of the course of the Tista River, which is also regarded as one of the major causes of shifting from the Old Brahmaputra to the present Jamuna, (2) the present Jamuna has taken its course along a depression running almost straight from north to south, which means that the slope of the Jamuna is excessively steep compared with a large river such as the Ganges and (3) the length of the river in this alluvial plain is too short compared with the magnitude of discharge and sediment load.

A braided river is presumed to be transformed eventually to a meandering one in the very remote future. The present Jamuna River also must have a similar nature. Even if the Jamuna should have this nature a state of meanering will not be encountered within 100 or 200 years, because it is only less than 200 years since the Old Brahmaputra shifted its course to the present Jamuna. Therefore, the construction of the bridge should be planned on the premise of braiding of the river.

In this river, the braiding produces cliffy banks almost on the whole length of the river at least within the land of Bangladesh. The space between the both cliffy banks is regarded as an effective width for flood flow.

Notwithstanding the both banks form cliffs, such clayey bank as seen at Sara on the Ganges is not found in the Jamuna River. In other words, any portion of the bank has no resisting power to erosion. This means that any portion of the bank cannot be fixed without any artifical protection. This holds not only at loops of braiding but also at nodes.

The Japanese Prefeasibility Study Team recommended the four nodes of braiding as sites to be proposed for bridge crossing. This is certainly appropriate since the nodes stand at the present places at least for about a hundred years. However, this is only based upon statistics and it is very regrettable to say that, at the present stage, we cannot elucidate the reason why such nodes have been produced and stood for such a long time as a hundred years.

Although it is very appropriate to choose a node as a bridge site, we can find no guarantee that these nodes will forever stand at the same places without any change of their forms. On the contrary, the fact is that there occurs incessant erosion at banks even at the nodes of braiding; it is a well-known fact that severe erosion occurs at the bank of Sirajganj and costly protection works are carried out every year. Therefore, even in case a bridge is spanned at such sites from one bank to the opposite, some revetment works will be inevitable so as to protect the abuttments of the bridge.

Moreover, in every highwater season, flood water always overflows over the both banks and the lowlying land which is located between the two Barinds is inundated irrespective of the river and the land. This phenomenon always facilitate not only spilling to tributaries but also incessant erosion at banks. In this meaning, this river can be called a river having no banks. Therefore, in case of constructing a bridge, the abuttments and the approches should be designed so as not to be destroyed no matter what erosion may occur at banks and no matter which course the thalweg may take.

For this purpose, two artificial banks with revetments and two closing dikes connected with the banks are required with a view to making the flood flow always run through a definite channel, namely, through a space between the two banks. In other words, the course of the flood flow should be fixed beteen the two banks by means of guiding function of the banks as well as by producing dead-water zones by the two closing dikes which can also be used for the road approaches to the bridge. Whether a bridge is spanned over the whole river width or a smaller one, guide banks are required together with closing dikes.

In conclusion, guide banks are needed in the case of the Jamuna River as far as a bridge is not spanned over a quite long distance which far exceeds the whole river width.

2. River Width at Spanning Site.

As is seen in aerophotos, the width of braiding is about 5 km (3.1 mi) even at the four nodes which are proposed for bridge construction, while it is about 15 km (9.3 mi) at the loops. The ratio of contraction is about 0.33. Even at a node, waterways are not integrated into one main, but remarkable braiding still remains. According to the reason mentioned in the previous section, guide bank system was adopted for the purpose of both guiding of flood flow and protection of river banks. In spanning across the river, several lengths must be considered from the viewpoint of construction costs of both river training works and bridge building. Hence, three kinds of river width were considered at each site.

(1) Type-A

It seems in the case of Hardinge Bridge that a width of about one mile was adopted as the total span in consideration of the width of a main channel in the dry season. In the case of the Jamuna, a width less than about 4,000 m (13,100 ft) is undesirable as mentioned in Chapter VI. However, a width of 2,000 m (6,600 ft) was taken into consideration merely as one case for counting the cost of river training works. We call this Type-A.

(2) Type-B

As already mentioned, a length of about 4,000 m (13,100 ft) is necessary as a standard minimum width. Adding a total length required for piers and losses due to them, a width of 4,200 m (13,800 ft) was taken as Type-B.

(3) Type-C

The width between both river banks (cliffs) at each proposed site was taken as Type-C, which is shown in the following.

| | |
|---|---|
| 5,200 m(17,000 ft) | at Nagarbari site |
| 5,600 m(18,400 ft) | at Sirajganj site |
| 5,200 m(17,000 ft) | at Gabargaon site |
| 5,600 m(18,400 ft) | at Bahadurabad site |

3. Alignment of Guide Banks.

R. R. Gales proposed in his paper (118 GB) alignments and lengths of guide banks necessary to lead flood flow toward between them to avoid damages to the approaches and protect piers by making the flow pass the bridge axis as uniformly as possible. The alignments and the lengths are shown in Fig. 7-1. In this figure, Gales uses the words of permanent banks. However, these words do not seem to be quite suitable in the case of the Jamuna, although they may have been reasonably used in the case of the Ganges because of existence of clayey bank at least at Sara.

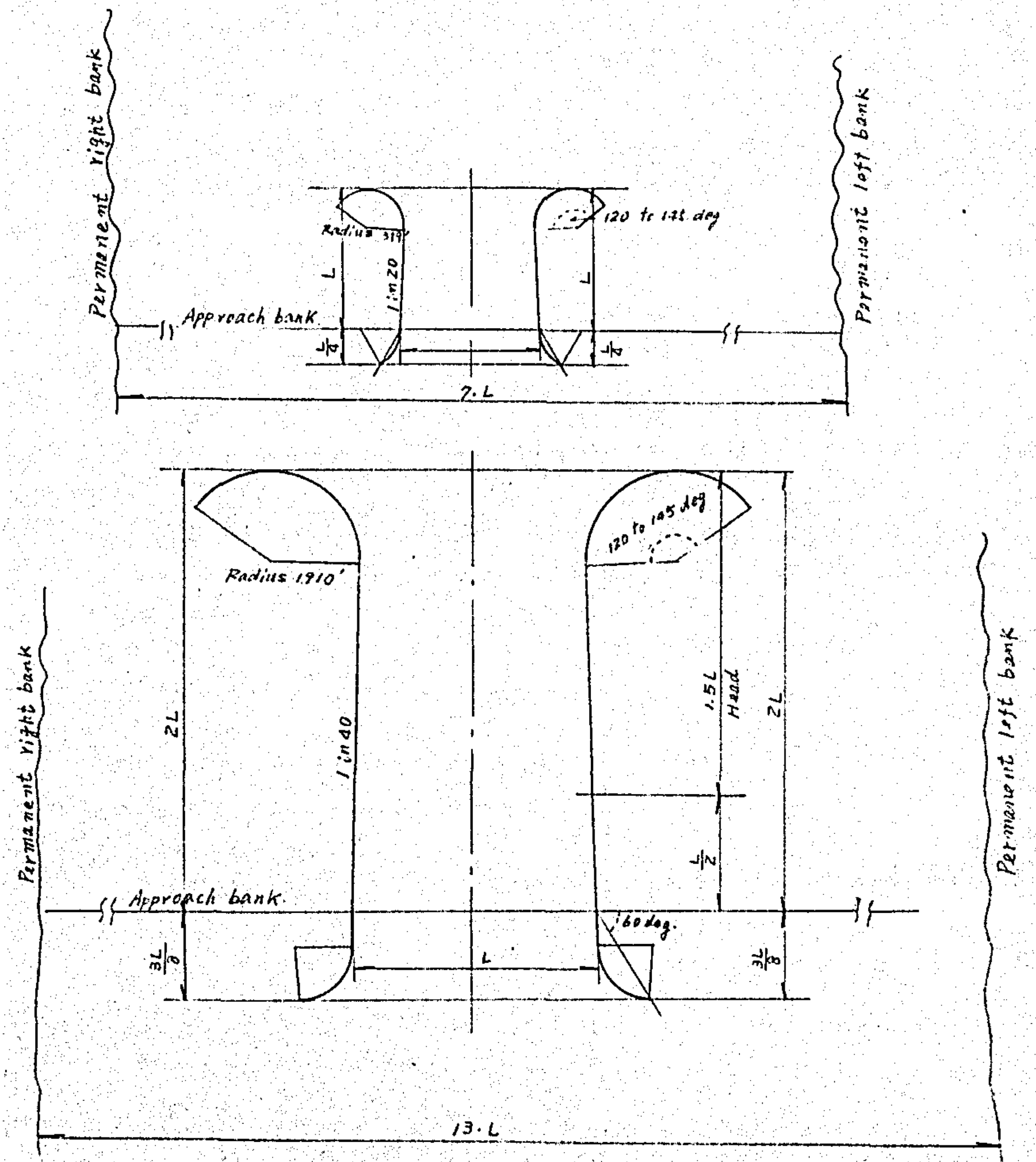
If we assume in the present case that the distance between two permanent banks is a width in which a river channel is moving freely, it is estimated at about 20 km according to the aerophotographs taken in the 1970/71 dry season. (12.4 mi)

Fig. 7-1 indicates that the length of guide bank upstream from the bridge axis should be taken two times as long as the distance between the two guide banks in case the permanent-bank width is thirteen times as wide as the guide-bank width and a length equal to the guide-bank width is required in case the permanent-bank width is seven times as wide as the guide-bank width. For an intermediate, interpolation can be made.

In the present case, Type-A alone lies inbetween. So the length of the guide banks upstream from the bridge axis was estimated at 3,000 m by means of interpolation. But the other three cases lie outside the range. For these cases, 3,000 m was taken irrespectively of the guide-bank width, because the same state of flow as expected in Type-A may occur along either of the two banks and, in our judgement, this length will be necessary to keep the function of guidance.

In regard to head of the guide bank, a shape of circular arc was taken in consideration of bending of river channels and the radius of the head arc was taken at 627 m (1910 ft) according to Gales' proposal. In case the design guide-bank width is 2,000 m, a straight-line length of 250 m was added to the outer edge of the head arc and, in the other cases, the length of the arc was limited within the inner angle of 120°.

Fig. 7-1 Dimension of Guide Bank
(by Gales)



Design length of guide bank downstream from the bridge axis was calculated at 750 m for the guide-bank width of 2,000 m according to the standard 3L/8 as shown in Fig. 7-1. Since there is no reason that the length can be shortened than the above for the other cases, it was decided that the length of 750 m was applied to all cases. Also the plane shapes shown in Fig. 7-1 were applied to any of them. Thus the alignment of guide banks was designed as shown in Fig. 7-2 from 1 to 4 and the total lengths of guide banks are shown in the following.

5,495 m (18,020 ft) on one side, 10,995 m (360,50 ft) on both sides for guide-bank width 2,000 m. 4,634 m (15,190 ft) on one side, 9,268 m (30,390 ft) on both sides for other guide-bank widths.

Further in future, in making detail designs, it is desirable that hydraulic model tests will be conducted of the alignment and length of guide banks.

4. Standard Cross Section of Guide Banks.

Standard cross section of guide banks must be designed on the basis of the following studies.

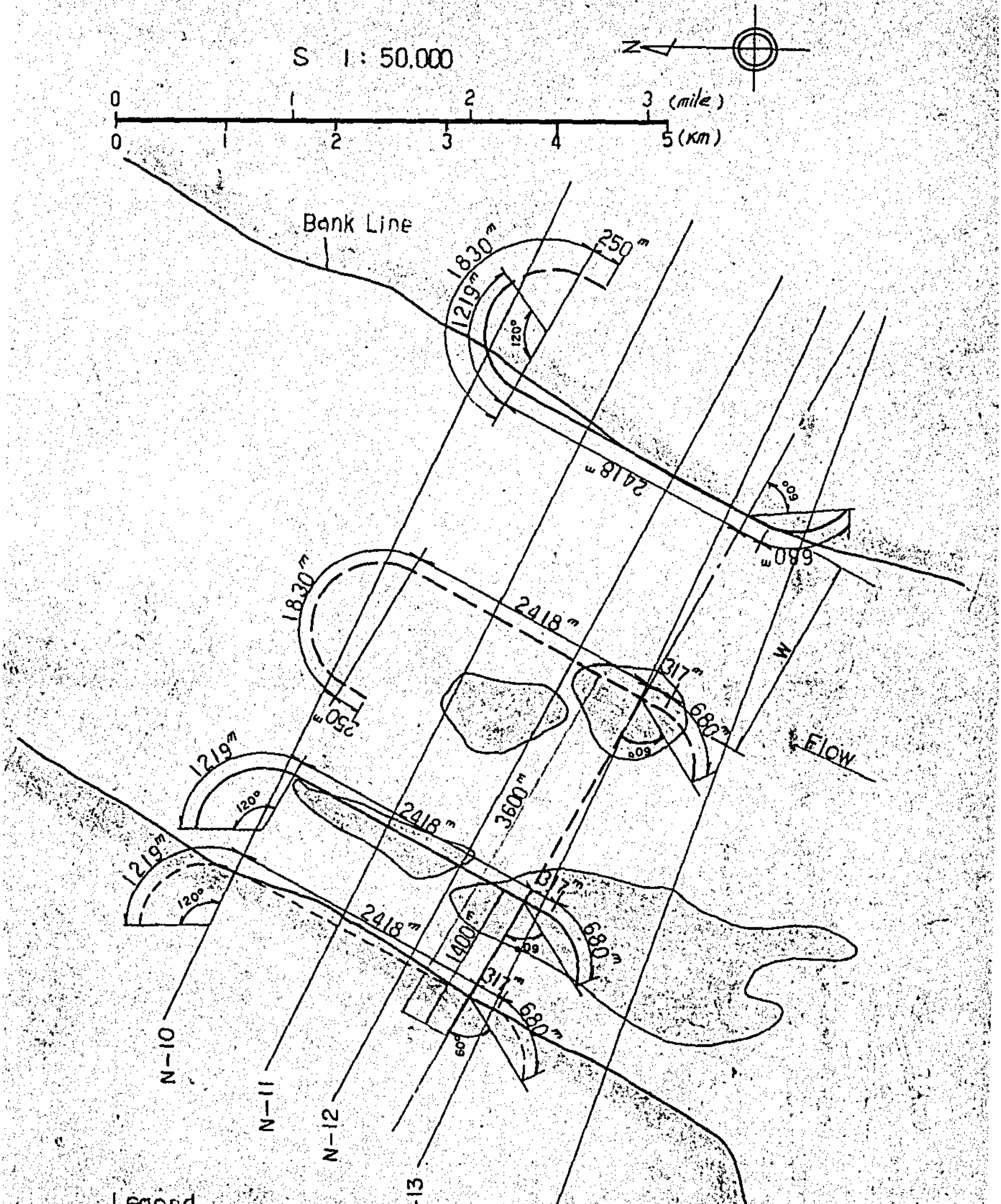
a. Height of bank; high water level corresponding to the design descharge.
b. Freeboard; run-up of wind waves, variation of river bed, consideration of safety for deviation of design discharge, etc.
c. Crown width; seepage line in embankment and width required for construction and maintenance works.
d. Slope gradient; stability of embankment and slope protection.
e. Length of apron; maximum probable scour in constriction due to guide banks.

(1) Design high water level.

i. Cross sections available.

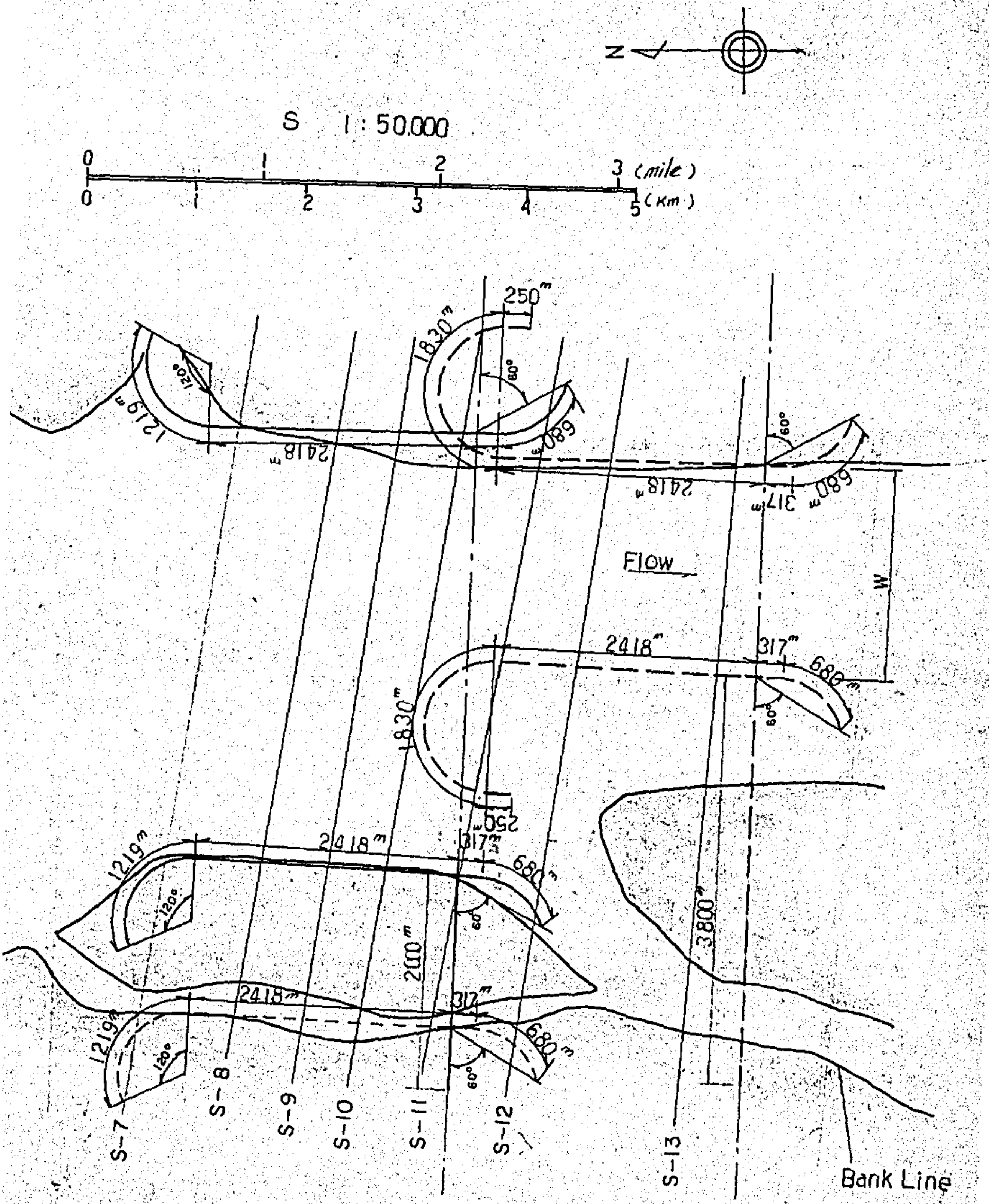
Cross sections of the Jamuna River from the confluence of the Tista

Fig.7-2-1 Nagarbari Site




Legend
Type A (W=2000m, L=5495m)
Type B (W=4200, L=4634)
Type C (W=5200, L=4634)
W: Width of Water Channel L: Length of Guide Bank

Fig. 7-2-2 Sirajganj Site



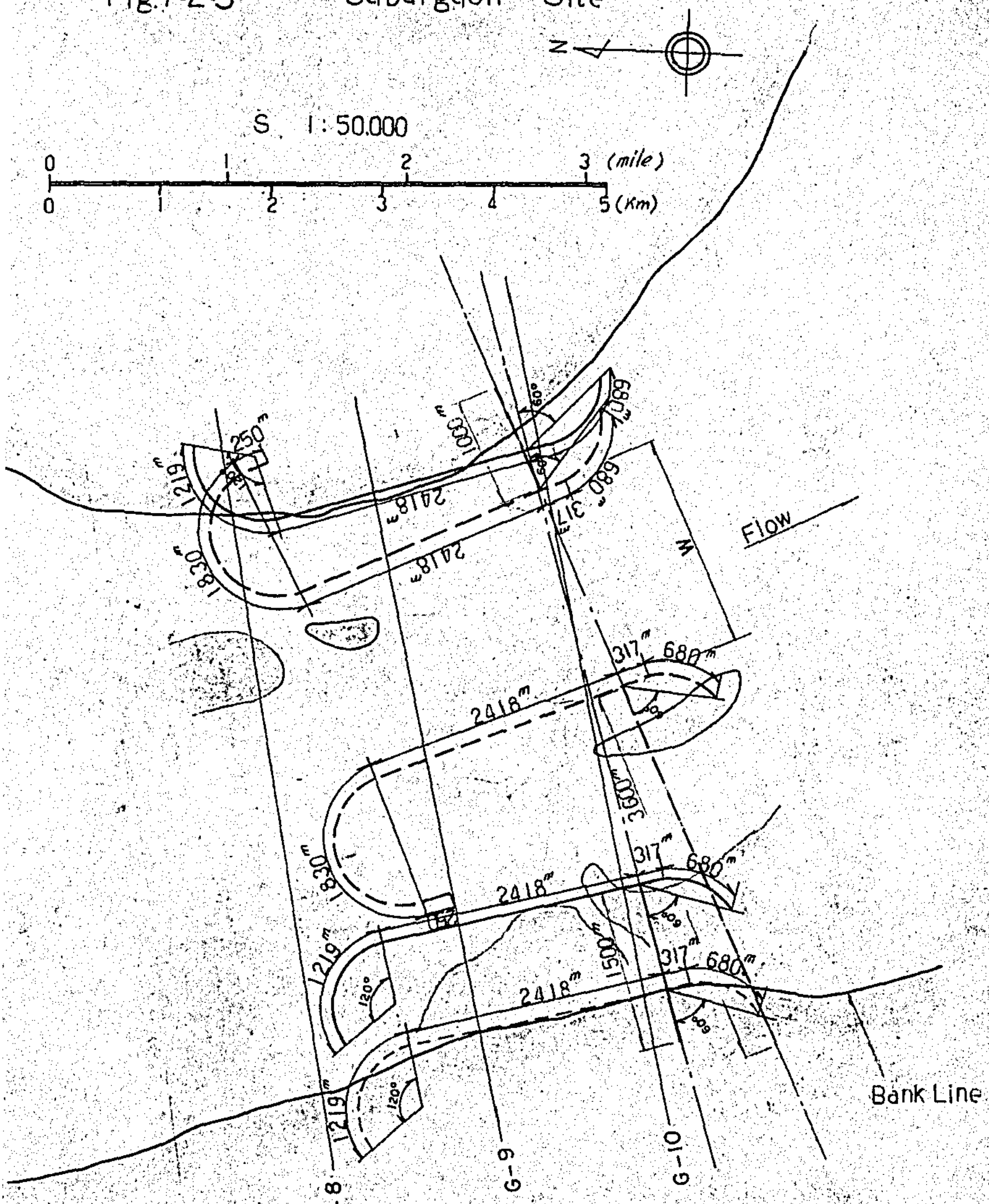

Legend

Type A (W=2000m, L=5495m) W: Width of Water Channel

Type B (W=4200, L=4634) L: Length of Guide Bank

Type C (W=5600, L=4634)

Fig.7-2-3 Gabargaon Site




Legend

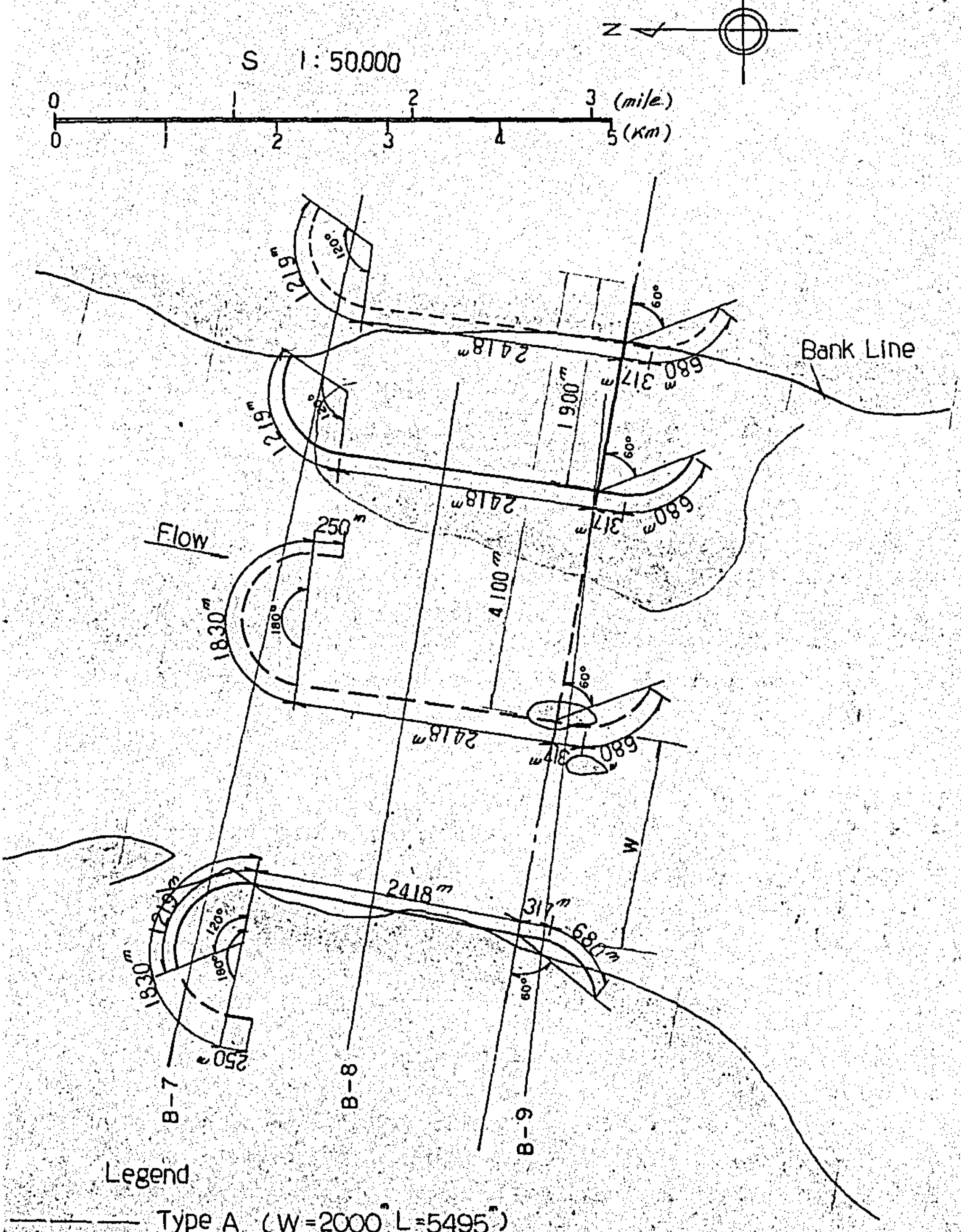
Type A (W=2000m, L=5495m)

Type B (W=4200, L=4634)

Type C (W=5200, L=4634)

W: Width of Water Channel, L: Length of Guide Bank

Fig. 7-2-4 Bahadurabad Site




Legend

— — — Type A ( W=2000m, L=5495m )

———— Type B ( W=4200, L=4634 )

- - - - - Type C ( W=5600, L=4634 )

W : Width of Water Channel L : Length of Guide Bank to that of the Ganges have been surveyed by BWDB every dry season since 1964/65 water year at intervals of 8 mi in 1964/65 and 4 mi after 1965/66. Further, in 1973, cross-section surveying within each stretch of 20 km including the proposed bridge axis was made by the Japanese surveying team in the period of falling stage of flood from September to October. Interval between sections at this surveying was about 1 km for Nagarbri and Sirajganj sites and about 2 km for Gabargaon and Bahadurabad sites.

ii. Preliminary study.

(i) Treatment of river section for water-level calculation.

Cross section of the river is generally devided into several channels in the dry season. However, for the convenience of calculation, equivalent cross section was considered integrating them into one channel because it was judged that the calculation of water level would be scarcely affected by this approximation. Thus in calculating the design high water level,

a. effective river width at ground level was determined by examining shapes of cross sections and plane shapes of the river on aerophotographs,

b. it was assumed that discharge at a water stage above ground level is confined within two embankments to be considered at the above-mentioned positions, and widths of chars above ground level were excluded from effective river width, because it was judged that discharge through these widths would be so small that it would scarecely affect the discharge through the whole water area by reason water depth on these chars would be very small and this treatment would give a little higher water stage.

(ii) Calculation of water profile along the river.

Water profile along the whole stretch of the Jamuna was calculated by uniform-flow equation by reason (1) mean water depth below ground level is almost constant over the whole stretch, (2) slope of the river is very gentle, and (3) intervals of about 4 mi are unsuitable for the calculation of non-uniform flow.

At first, discharge allocation was made in consideration of diversion to the Old Brahmaputra and the Dhaleswari Rivers on the basis of the Bahadurabad flood discharge on July 28, 1970 which was the largest one in recent years, and then, on the basis of this discharge allocation, water profile was calculated using the cross sections surveyed by BWDB in the dry season of 1969/70, which is shown in Fig. 7-3, in which water level measured on July 28 are also given. In this calculation, measured water profile and a roughness coefficient of 0.02 were applied.

# Fig. 7-3 Water Level Calculated by Manning's Formula

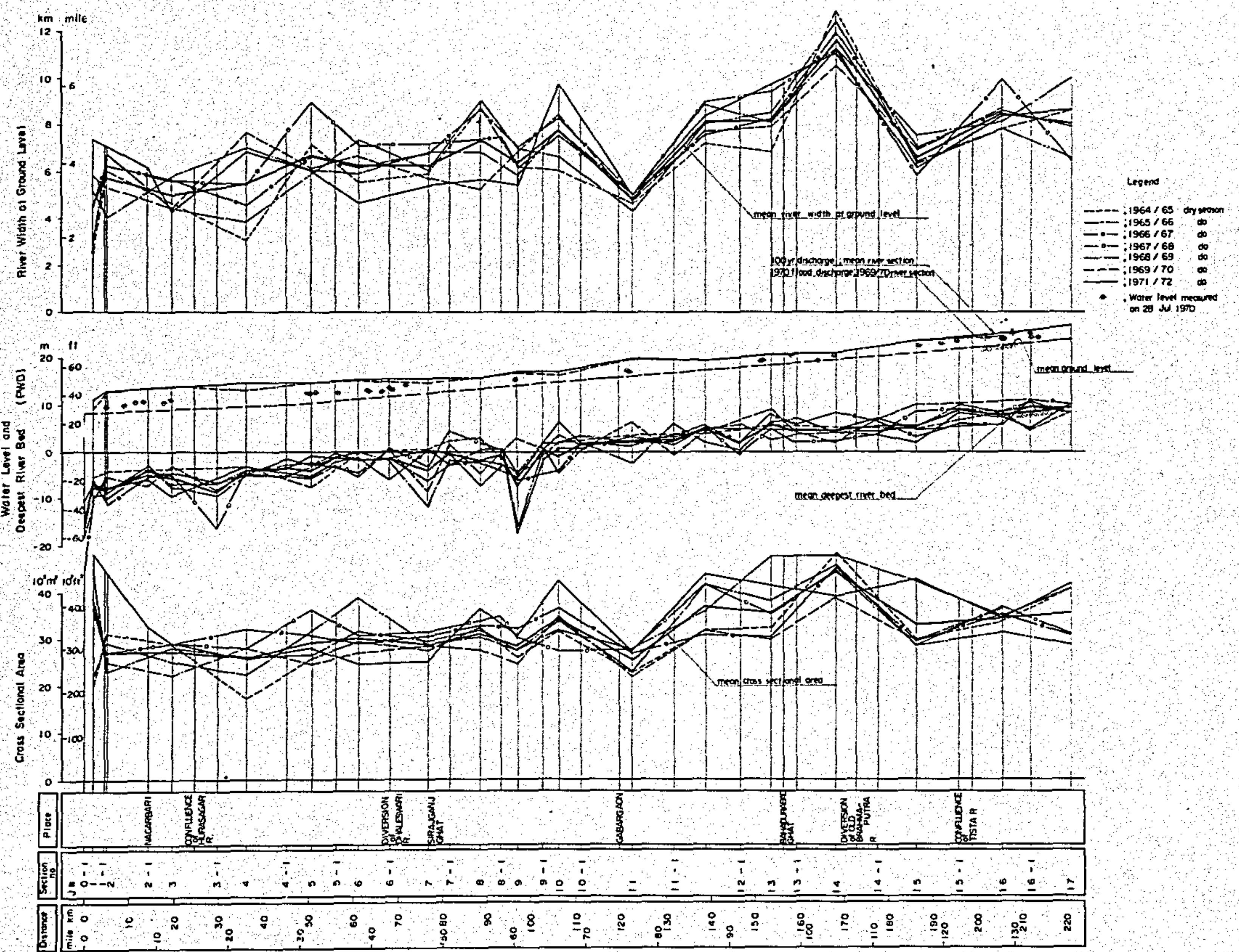


Table 7-1 Mean Velocity along the River Course
(1970 Maximum Flood and 1969/70 cross sections)

| Sect. No. J# | Water stage (m, PWD)(ft, PWD) | Water area ($10^3 m^2$) | Discharge ($m^3/s$) | Mean velocity (m/s)(ft/s) | Remarks |
|---|---|---|---|---|---|
| 1 | 9.00 (29.51) | 41.52 | 70,660 | 1.70 (5.57) | n = 0.002 |
| 2 | 12.68 (41.57) | 53.13 | 70,660 | 1.33 (4.36) | |
| 3 | 14.07 (46.13) | 44.13 | 70,660 | 1.60 (5.25) | |
| 4 | 13.25 (43.44) | 55.25 | 70,660 | 1.28 (4.20) | |
| 5 | 14.83 (48.62) | 50.34 | 70,660 | 1.40 (4.59) | |
| 6 | 15.20 (49.83) | 52.06 | 70,660 | 1.36 (4.46) | |
| 7 | 14.93 (48.95) | 43.47 | 76,460 | 1.76 (5.77) | |
| 8 | 15.77 (51.70) | 42.19 | 76,460 | 1.81 (5.93) | |
| 9 | 16.96 (55.61) | 46.71 | 76,460 | 1.64 (5.38) | |
| 10 | 16.60 (54.43) | 51.24 | 76,460 | 1.49 (4.89) | |
| 11 | 19.86 (65.11) | 40.33 | 76,460 | 1.90 (6.23) | |
| 12 | 19.64 (64.39) | 49.18 | 76,460 | 1.56 (5.11) | |
| 13 | 21.10 (69.18) | 49.93 | 76,460 | 1.53 (5.02) | |
| 14 | 21.30 (69.84) | 56.05 | 80,260 | 1.43 (4.69) | |
| 15 | 23.67 (77.61) | 46.92 | 80,260 | 1.71 (5.61) | |
| 16 | 24.54 (80.46) | 51.73 | 80,260 | 1.55 (5.08) | |
| | | | | mean 1.57 (5.15) | |

Next, water profile for the design discharge (100-year flood) was calculated using average cross sections surveyed by BWDB since 1964/65, which is also shown in Fig. 7-3.

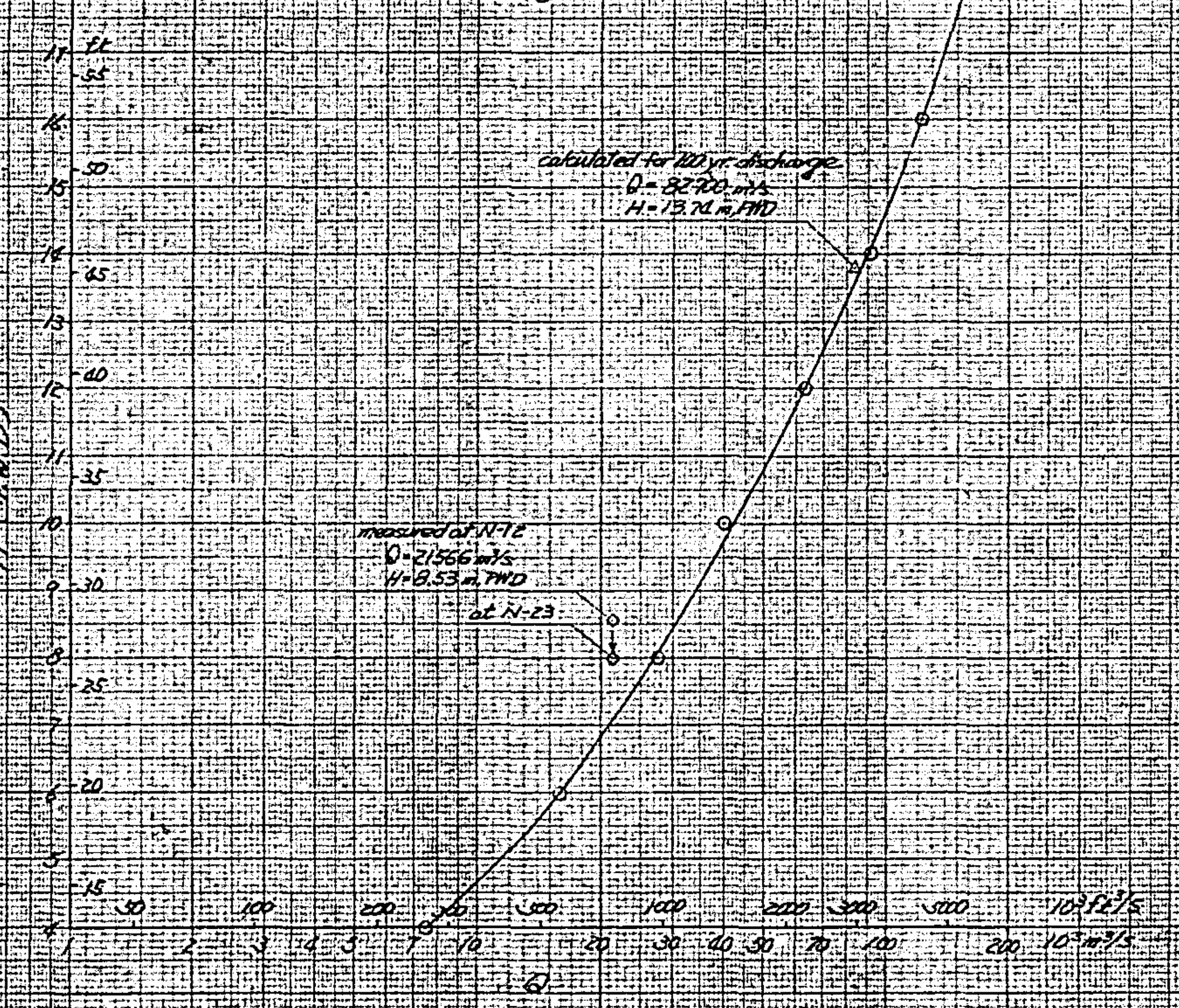
iii Design high water level

Water profile in case guide banks were constructed at the proposed site was calculated by nonuniform-flow equation using the cross sections surveyed by the Japanese surveying team at intervals of 1 km or 2 km. The conditions which the calculation was based upon are as follows.

a. Discharge : 100-year flood discharge.

b. Water level at the most downstream section : this water level was determined by rating curve which was made by uniform-flow equation making use of cross sections of lower 5-km stretch and water surface slope obtained in the preliminary study previously mentioned. The rating curves for the proposed four sites are shown in Figs. 7-4, in which the discharges measured by the surveying team and the water profile by the preliminary study for 100-year flood are also shown.

Fig 7-4-1 Rating Curve at Nagarbari



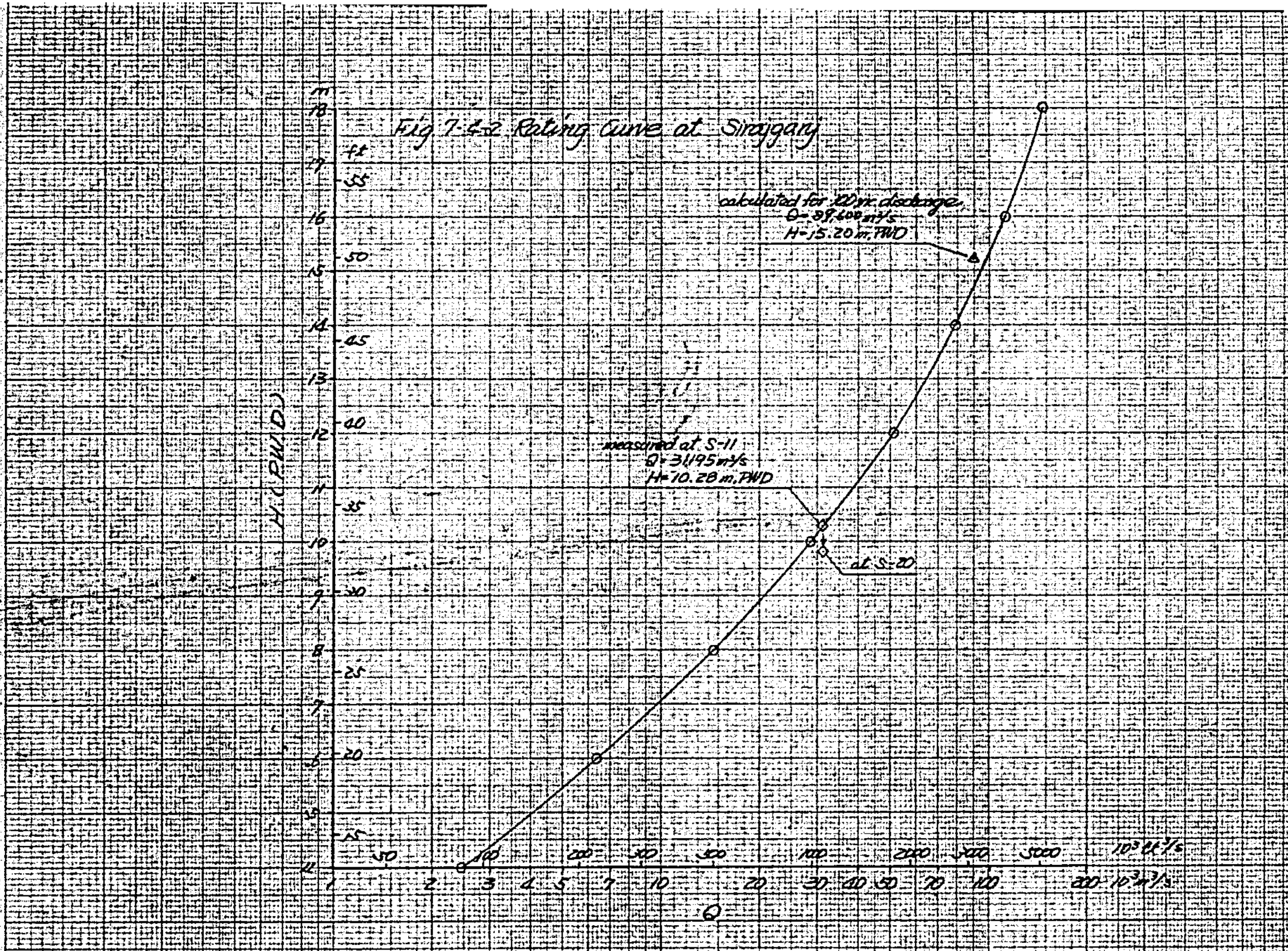


Fig 7-4-2 Rating Curve at Sirajganj
calculated for 100 m. discharge
Q=89,400 m³/s
H=15.20 m. PWD
measured at S-11
Q=31,195 m³/s
H=10.28 m. PWD
at S-20
H (PWD)
m
ft
Q
10³ ft³/s
10³ m³/s

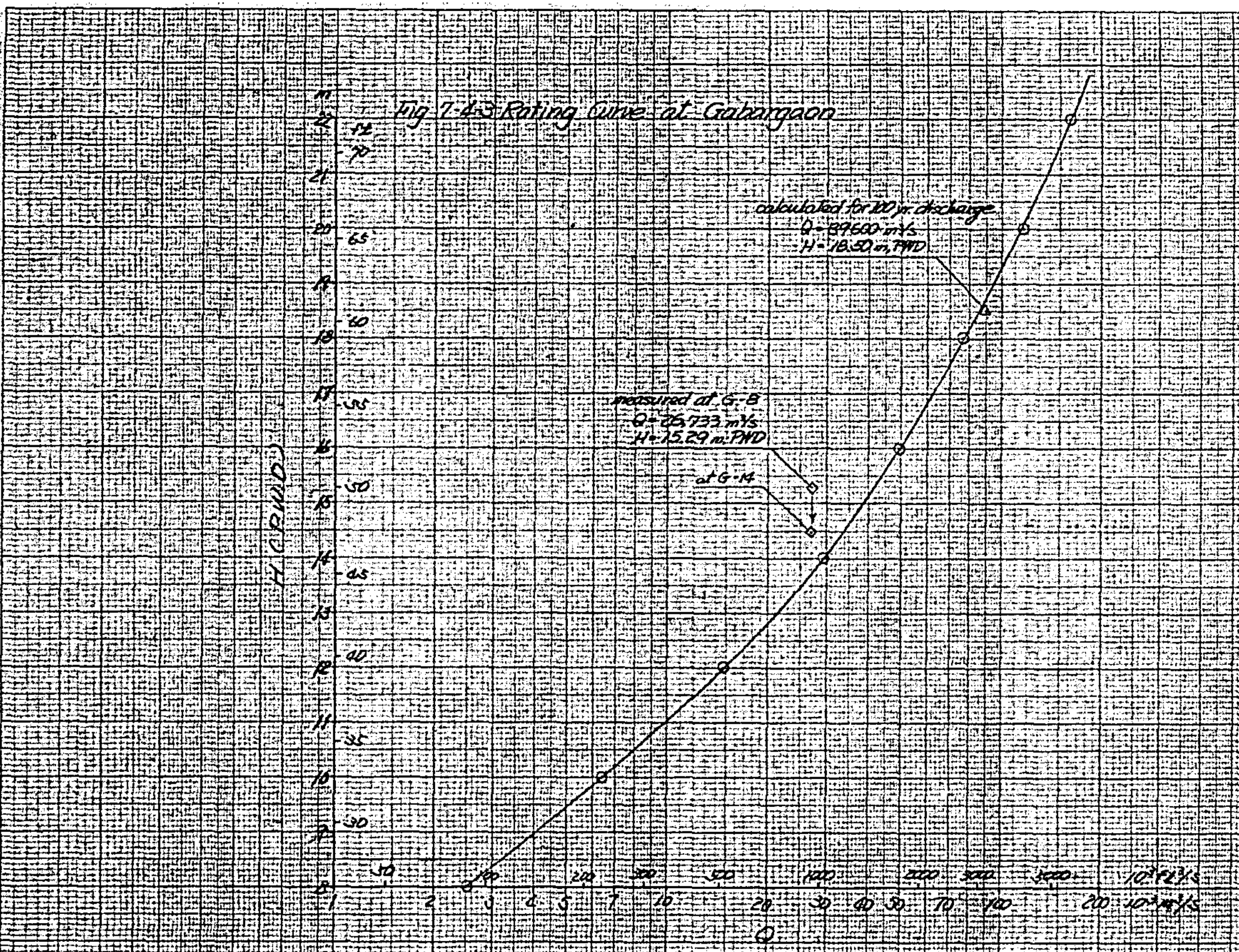



Fig 7-4-3 Rating Curve at Gabargaon

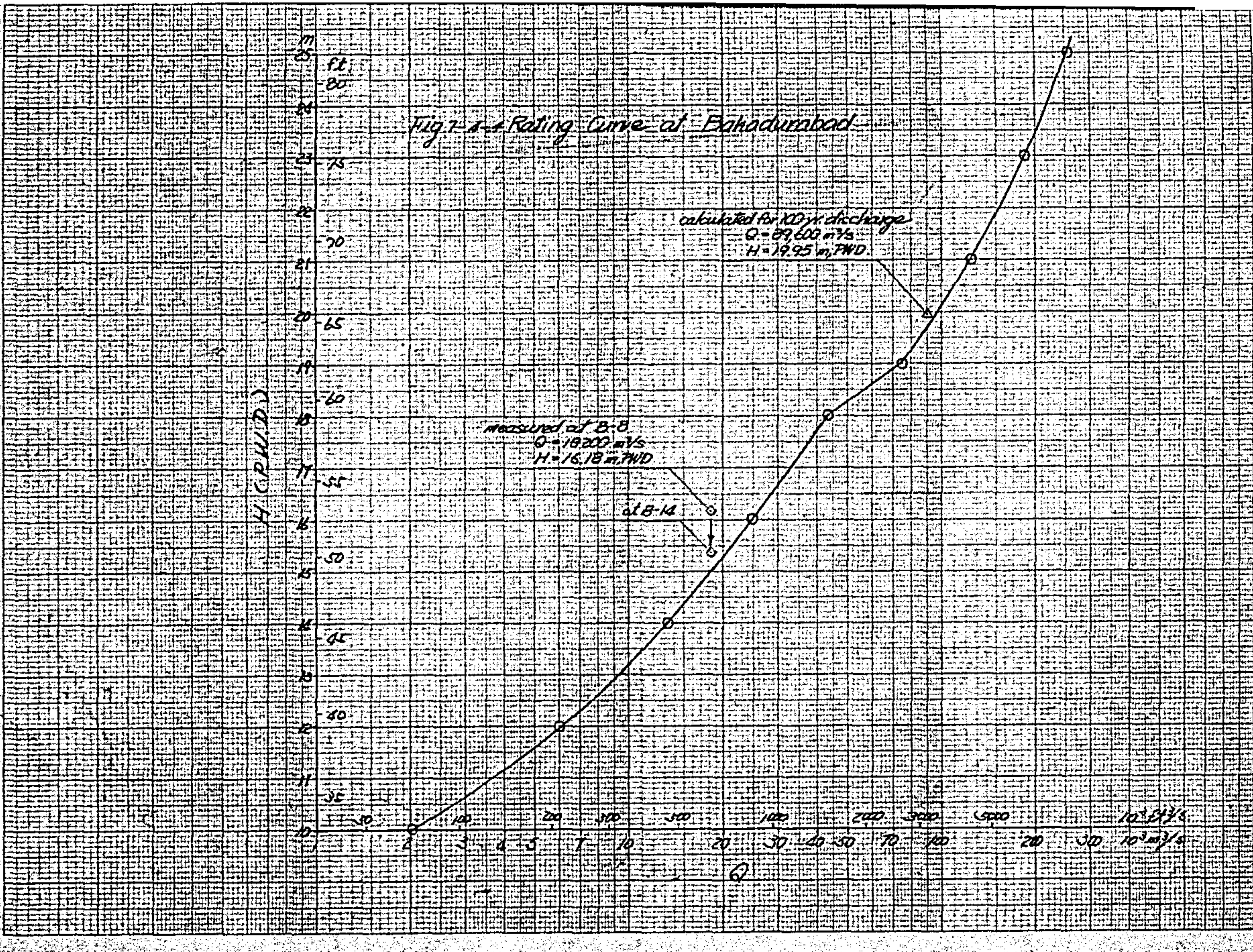



Fig.1-4-4 Rating Curve at Bahadurabad

c. Coefficient of roughness : n = 0.02.

d. Interval between cross sections for calculation : this interval was taken at about 500 m, that is, interpolation was made between two sections since the interval of 1 km or 2 km is too large for the calculation.

The results of calculation are shown in Table 7-2. On the basis of these results, the design high water level was determined as follow.

a. Nagarbari site.

DHWL = 14.01 m (45.94 ft), PWD at Sect. N-13

I = 1/25,400

b. Sirajganj site.

DHWL = 15.24 m (49.97 ft), PWD at Sect. S-11

I = 1/18,500

c. Gabargaon site.

DHWL = 19.44 m (63.74 ft), PWD at Sect. G- 8

I = 1/15,000

d. Bahadurabad site

DHWL = 20.86 m (68.40 ft), PWD at Sect. B- 8

I = 1/15,000

The design high water levels are also shown in Table 7-2.

Table 7 - 2 - 1 Calculation of Design Water Level

Nagarbari - site

| Section NO. | Distance (Km) between sections | Distance (Km) cumulative | Average G.H. (m, PWD) | Calculated W.L. (m, PWD) | Design H.W.L. (m, PWD) | L.W.L. (m, PWD) |
|---|---|---|---|---|---|---|
| N-23 | 0 (km) | 0 (km) | 9.100 | 13.550 | 13.608 | 44.62 |
| 22 | 1.14 | 1.14 | 9.139 | 13.595 | 13.647 | 44.75 |
| 21 | 1.10 | 2.24 | 9.178 | 13.638 | 13.695 | 44.91 |
| 20 | 0.90 | 3.14 | 9.209 | 13.668 | 13.730 | 45.02 |
| 19 | 1.15 | 4.29 | 9.249 | 13.704 | 13.775 | 45.17 |
| 18 | 1.05 | 5.34 | 9.285 | 13.748 | 13.816 | 45.30 |
| 17 | 1.22 | 6.56 | 9.327 | 13.814 | 13.864 | 45.46 |
| 16 | 1.00 | 7.56 | 9.362 | 13.860 | 13.903 | 45.59 |
| 15 | 0.88 | 8.44 | 9.392 | 13.894 | 13.937 | 45.70 |
| 14 | 0.80 | 9.24 | 9.420 | 13.934 | 13.968 | 45.80 |
| 13 | 0.98 | 10.22 | 9.454 | 13.978 | 14.006 | 45.93 |
| 12 | 0.68 | 10.90 | 9.478 | 13.999 | 14.033 | 46.01 |
| 11 | 0.80 | 11.70 | 9.505 | 14.030 | 14.064 | 46.12 |
| 10 | 1.10 | 12.80 | 9.543 | 14.092 | 14.107 | 46.26 |
| 9 | 1.22 | 14.02 | 9.586 | 14.123 | 14.154 | 46.41 |
| 8 | 1.05 | 15.07 | 9.622 | 14.135 | 14.195 | 46.55 |
| 7 | 1.01 | 16.08 | 9.657 | 14.162 | 14.235 | 46.68 |
| 6 | 0.92 | 17.00 | 9.689 | 14.210 | 14.271 | 46.79 |
| 5 | 1.18 | 18.18 | 9.730 | 14.233 | 14.317 | 46.95 |
| 4 | 1.01 | 19.19 | 9.715 | 14.293 | 14.356 | 47.07 |
| 3 | 0.75 | 19.94 | 9.791 | 14.370 | 14.385 | 47.17 |
| 2 | 0.76 | 20.70 | 9.817 | 14.391 | 14.415 | 47.27 |
| 1 | 0.95 | 21.65 | 9.850 | 14.403 | 14.452 | 47.39 |

Sirajganj - site

| Section NO. | Distance (Km) between sections | Distance (Km) cumulative | Average G.H. (m, PWD) | Calculated W.L. (m, PWD) | Design H.W.L. (m, PWD) | L.W.L. (m, PWD) |
|---|---|---|---|---|---|---|
| S-20 | 0 (km) | 0 (km) | 10.650 | 14.620 | 14.700 | 48.20 |
| 19 | 1.07 | 1.07 | 10.728 | 14.689 | 14.758 | 48.39 |
| 18 | 1.09 | 2.16 | 10.808 | 14.724 | 14.877 | 48.78 |
| 17 | 0.96 | 3.12 | 10.879 | 14.764 | 14.869 | 48.75 |
| 16 | 1.14 | 4.26 | 10.962 | 14.813 | 14.930 | 48.95 |
| 15 | 1.18 | 5.44 | 11.049 | 14.862 | 14.994 | 49.16 |
| 14 | 1.01 | 6.75 | 11.145 | 14.919 | 15.065 | 49.39 |
| 13 | 1.28 | 8.03 | 11.238 | 15.008 | 15.134 | 49.62 |
| 12 | 1.28 | 9.31 | 11.332 | 15.069 | 15.203 | 49.85 |
| 11 | 0.66 | 9.97 | 11.380 | 15.108 | 15.239 | 49.96 |
| 10 | 0.71 | 10.68 | 11.432 | 15.128 | 15.277 | 50.09 |
| 9 | 0.54 | 11.22 | 11.472 | 15.166 | 15.302 | 50.19 |
| 8 | 0.57 | 11.79 | 11.514 | 15.187 | 15.337 | 50.29 |
| 7 | 0.88 | 12.67 | 11.578 | 15.278 | 15.385 | 50.44 |
| 6 | 1.27 | 13.94 | 11.621 | 15.343 | 15.454 | 50.67 |
| 5 | 1.11 | 15.05 | 11.753 | 15.442 | 15.514 | 50.87 |
| 4 | 1.02 | 16.07 | 11.827 | 15.512 | 15.569 | 51.05 |
| 3 | 0.90 | 16.97 | 11.893 | 15.586 | 15.617 | 51.20 |
| 2 | 1.09 | 18.06 | 11.973 | 15.605 | 15.676 | 51.40 |
| 1 | 1.05 | 19.11 | 12.050 | 15.653 | 15.733 | 51.58 |

# Table 7 - 2 - 2 Calculation of Design Water Level

## Gabargaon - site

| Section NO. | Distance (Km) between sections | Distance (Km) cumulative | Average G.H. (m, PWD) | Calculated W.L. (m, PWD) | Design HWL (m, PWD) | Design HWL (ft, PWD) |
|---|---|---|---|---|---|---|
| G-14 | 0 (km) | 0 (km) | 15.350 | 18.710 | 18.950 | 62.14 |
| 13 | 2.04 | 2.04 | 15.512 | 18.879 | 19.086 | 62.58 |
| 12 | 1.64 | 3.68 | 15.641 | 18.989 | 19.195 | 62.94 |
| 11 | 1.87 | 5.55 | 15.789 | 19.083 | 19.320 | 63.35 |
| 10 | 1.81 | 7.36 | 15.933 | 19.202 | 19.441 | 63.75 |
| 9 | 1.52 | 8.88 | 16.053 | 19.294 | 19.542 | 64.08 |
| 8 | 1.30 | 10.18 | 16.156 | 19.381 | 19.628 | 64.36 |
| 7 | 1.30 | 11.48 | 16.259 | 19.464 | 19.715 | 64.65 |
| 6 | 1.10 | 12.58 | 16.364 | 19.557 | 19.788 | 64.89 |
| 5 | 1.34 | 13.92 | 16.452 | 19.640 | 19.678 | 65.18 |
| 4 | 1.56 | 15.48 | 16.576 | 19.778 | 19.982 | 65.52 |
| 3 | 1.55 | 17.03 | 16.698 | 19.923 | 20.085 | 65.86 |
| 2 | 1.70 | 18.73 | 16.833 | 20.034 | 20.198 | 66.23 |
| 1 | 1.48 | 20.21 | 16.950 | 20.126 | 20.297 | 66.55 |

## Bahadurabad - site

| Section NO. | Distance (Km) between sections | Distance (Km) cumulative | Average G.H. (m, PWD) | Calculated W.L. (m, PWD) | Design HWL (m, PWD) | Design HWL (ft, PWD) |
|---|---|---|---|---|---|---|
| B-14 | 0 (km) | 0 (km) | 17.300 | 19.700 | 20.300 | 66.56 |
| 13 | 2.00 | 2.00 | 17.455 | 19.987 | 20.433 | 67.00 |
| 12 | 2.12 | 4.12 | 17.620 | 20.233 | 20.575 | 67.47 |
| 11 | 1.28 | 3.40 | 17.719 | 20.344 | 20.660 | 67.74 |
| 10 | 1.59 | 6.99 | 17.843 | 20.437 | 20.766 | 68.09 |
| 9 | 1.40 | 8.39 | 17.951 | 20.510 | 20.859 | 68.40 |
| 8 | 1.44 | 9.83 | 18.063 | 20.673 | 20.955 | 68.71 |
| 7 | 1.36 | 11.19 | 18.169 | 20.805 | 21.046 | 69.01 |
| 6 | 0.89 | 12.08 | 18.238 | 20.931 | 21.105 | 69.20 |
| 5 | 1.38 | 13.46 | 18.345 | 21.031 | 21.197 | 69.51 |
| 4 | 1.82 | 15.28 | 18.486 | 21.202 | 21.318 | 69.90 |
| 3 | 1.93 | 17.21 | 18.636 | 21.349 | 21.447 | 70.32 |
| 2 | 1.45 | 18.66 | 18.748 | 21.472 | 21.544 | 70.64 |
| 1 | 1.31 | 19.97 | 18.850 | 21.566 | 21.631 | 70.93 |

(2) Freeboard.

Freeboard should be provided in consideration of variation of design discharge, variation of river bed, wind wave, etc. Five-foot freeboard has been taken in the case of the right flood embankment of the Jamuna. Therefore, it is judged that about three-meter freeboard is sufficient also in this case. However, in order to make sure of it, run-up height of wind wave was, at least examined as follows.

i. Height of wind wave.

(i) Wind speed.

The maximum wind speed in the past in Bangladesh is shown in Fig. 7-5 and Table 7-3. It is evident from this figure that higher maximum wind speed occurs on the southeastern coast such as Chittagong and Cox's Bazar, while it decreases to the west and to the inland, for instance, 13.4 m/s at Sirajganj, 11.8 m/s at Bogra and 8.6 m/s at Rangpur. In consideration of these values, wind speed for examination of wave was determined at 15 m/s(49.2 ft/s).

(ii) Fetch and water depth.

10-km fetch and 10-m depth were adopted in consideration of topographic features.

(iii) Wave height and period.

On condition that wind speed $u_{10}$ = 15 m/s, fetch F = 10 km and water depth h = 10 m, wave height $H_{1/3}$ was calculated by Bretschneider' theory on shallow waves (119 GB). Thus the dimensions of wave are as follows.

Wave height ($H_{1/3}$) : $gH_{1/3}/U_{10}^2 = 0.046 \therefore H_{1/3} = 0.946$ m

Wave period ($T_{1/3}$) : $T_{1/3} = 3.86\sqrt{H_{1/3}} = 3.75$ sec.

Wave length (L) : $L = \frac{gT_{1/3}^2}{2\pi}\tanh\left(\frac{2\pi h}{L}\right) \therefore L = 21.9$ m

Wave celerity (c) : $C = L/T_{1/3} = 5.84$ m/s

Wave steepness ($H_{1/3}/L$) : $H_{1/3}/L = 0.043$

When number of waves is denoted by N, relation between significant wave height $H_{1/3}$ and the maximum wave height of N waves is given by Rayleigh's distribution in the following.

| | | | | | | |
|---|---|---|---|---|---|---|
| N | 50 | 100 | 200 | 500 | 1,000 | 10,000 |
| $H_{max}/H_{1/3}$ | 1.42 | 1.53 | 1.64 | 1.77 | 1,86 | 2,15 |

When N is large, the following equation holds approximately.

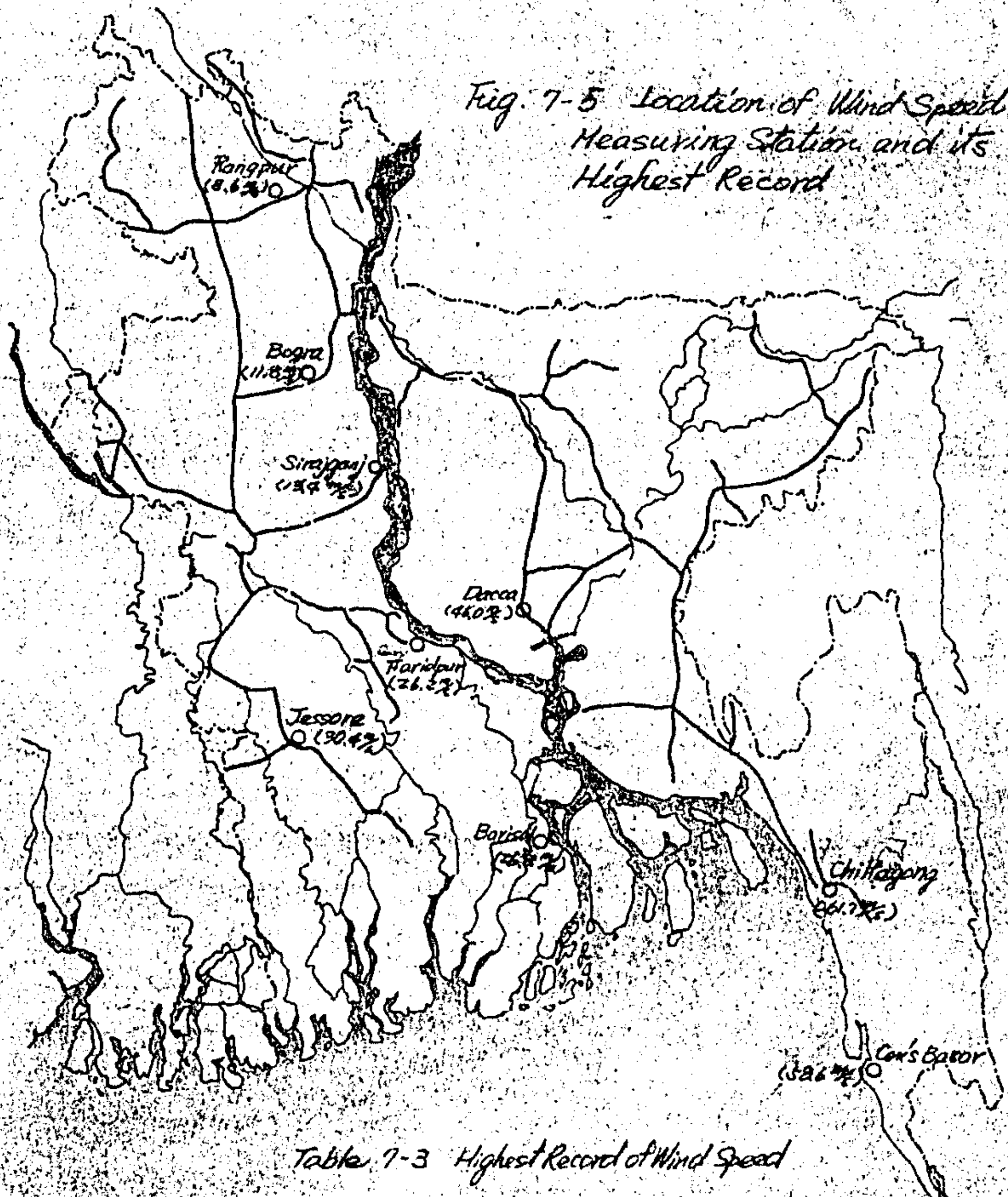



Fig. 7-5 Location of Wind Speed Measuring Station and its Highest Record

Table 7-3 Highest Record of Wind Speed

| Station | Period of available data | Highest wind speed (m/s) | Highest wind speed (knot) |
|---|---|---|---|
| Rangpur | 1960 ~ 1972 | 8.6 | 17 |
| Faridpur | " " | 26.2 | 51 |
| Sirajganj | " " | 13.4 | 26 |
| Bogra | " " | 11.8 | 23 |
| Dacca | 1955 ~ 1970 | 46.0 | 89 |
| Chittagong | " " | 61.7 | 120 |
| Cox's Bazar | 1960 ~ 1970 | 58.6 | 114 |
| Barisal | 1964 ~ 1970 | 26.8 | 52 |
| Jessore | 1960 ~ 1969 | 30.4 | 59 |

$$H_{max}/H_{1/3} = 1.07\sqrt{\log_{10}N}$$

If we assume that wind of 15 m/s continues to blow for one hour, the number of waves will be 960. Hence

$$H_{max} = 1.85 \times 0.946 = 1.75 \text{ m } (5.74 \text{ ft})$$

(iv) Run-up height.

Run-up height of wave on the slope of guide bank was estimated by Savil's study (119 GB). In case slope gradient of the bank is 1 : 3, ratio of run-up height R to that of corresponding deep-water wave $H_o$ is

1.5 for smooth slope, and
0.64 for slope protected by wave-absorbing works such as special precast concrete blocks.

Since the wave height 0.946 m is nearly equal to that of deep-water wave, the run-up height will be

$$R = 1.5 \times 0.946 = 1.42 \text{ m } (4.66 \text{ ft}) \quad \text{for significant wave}$$
$$R = 1.5 \times 1.75 = 2.63 \text{ m } (8.62 \text{ ft}) \quad \text{for maximum wave}$$

in case of smooth slope. Actual run-up height will be less than the above since bank slope in practice will be protected by stones or concrete blocks.

ii. Freeboard.

From the view point of wave run-up alone, 2.63 m are sufficient for freeboard even in case of maximum wave and smooth slope. In the present case, however, 3 m (9.8 ft) was adopted as freeboard in consideration of variation of design flood and river bed, etc.

(3) Crown width.

Crown width of guide bank was determined at 10 m (32.8 ft) taking into consideration the convenience of construction works and maintenance after completion.

(4) Slope gradient.

Gradient of river-side slope of the guide bank was determined at 1:3 according to overall-apron system proposed by Gales, while gradient 1:2 was adopted for land-side slope keeping 1:3 as a whole providing with berms on the slope.

Rip-rup revetment with polyethylene mat was adopted as protection works for the river-side slope and protection by ployethylene mat and sodding was considered for the land-side slope. Weight of the stones on the river-side slope was examined by Hudson's formula in consideration of resistance to wave power. When we take unit weight of pitching stone at 2.65 $t/m^3$, unit weight of water at 1 $t/m^3$, wave height at 0.946 m and the value of constant for material covering the slope at 3.2, we get 52.2 kg (115.1 lb) as the required weight of one stone.

(5) Apron.

i. Mean water depth between guide banks.

(i) Equation for calculation.

When a river is locally contracted by guide banks, velocity will be increased and river bed lowered at the constriction. The amount of lowering can be estimated on condition that quantity of silt transportation is constant through both the constriction and the stretches up-and-downstream of it. When we solve simultaneously Manning's equation for uniform flow and exponential equation for silt transportation for a wide rectangular channel on condition that discharge and quantity of silt transportation are constant between two successive sections, we obtain the ratio of water depths of the two sections as given in the following,

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-(1-\frac{1}{p})\frac{6}{7}}$$

where p is the exponent in the equation of silt transportation

$$Q_s = \alpha B u_*^p$$

$Q_s$ = sediment load
$\alpha$ = constant
$p$ = exponent
$u_*$ = friction velocity

and

$B$, $H$ = channel width and water depth
$B_o$, $H_o$ = channel width and water depth at reference section.

The value of the exponent p is 3 in Sato-Kikkawa-Ashida's formula and 5 in Brown's formula. Hence the above equation of water-depth ratio is expressed by

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-4/7} = \left(\frac{B}{B_o}\right)^{-0.57}$$ : by Sato-Kikkawa-Ashida's formula for bed-load transport

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-24/35} = \left(\frac{B}{B_o}\right)^{-0.69}$$ : by Brown's formula for suspended load transport

On the other hand, E.M. Laursen proposes the following equations.

$$\frac{d_s}{H_o} = \left(\frac{B_o}{B}\right)^{0.59} - 1 \quad \text{for } u_*/w < 1/2 \quad \text{(traction)}$$

$$\frac{d_s}{H_o} = \left(\frac{B_o}{B}\right)^{0.64} - 1 \quad \text{for } u_*/w = 1 \quad \text{(transition)}$$

$$\frac{d_s}{H_o} = \left(\frac{B_o}{B}\right)^{0.69} - 1 \quad \text{for } u_*/w > 2 \quad \text{(suspension)}$$

where

$d_s$ = scoured mean depth = $H - H_o$
$u_*$ = friction velocity
$w$ = settling velocity of sediment particle.

In case the difference between the two velocities of the two sections contracted and noncontracted or the head loss due to contraction are small, we get

$$\frac{H}{H_o} = \frac{d_s}{H_o} + 1$$

Hence we obtain the following equations for the above.

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-0.59} \quad \text{for } u_*/w < 1/2$$

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-0.64} \quad \text{for } u_*/w = 1$$

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-0.69} \quad \text{for } u_*/w > 2$$

According to Section 3, Chapter II, mean diameter of bed material of the river is 0.018 cm at Nagarbari, Sirajganj and Gabargaon sites and 0.027 cm at Bahadurabad. The ratio of friction velocity to settling velocity is larger than 2 in any case of A, B, and C types of constriction for the design discharge of each site. In this case, Laursen's formula accords with Brown's. Thus the equation

$$\frac{H}{H_o} = \left(\frac{B}{B_o}\right)^{-0.69}$$

was applied to the present case.

(ii) Examination of applicability of the equation.

In order to examine the applicability of the above equation, it was first applied to simplified sections (wide rectangular sections) of the existing channel, in which $B_o$ and $H_o$ were respectively taken as width and water depth of the reference section at the lowest end of each stretch. In this calculation, there appeared some deviation from the existing river bed, although the results of calculation could express the existing state of the river in fairely good accord.

Next, water depth of reference section was revised so that calculated mean water depths might become almost equal to those of the existing river over its whole stretch. The results are shown in Table 7-4 and Figs. 7-6-1 to 4, which indicate very good accord between them. Hence it proves that the equation is applicable to the calculation of lowering of river bed at the constriction due to guide banks.

(iii) Lowering of river bed at constriction due to guide banks.

Lowering of river bed at constriction due to guide banks was calculated for each of the previously mentioned constriction types using the reference section determined in the above Article. In this case, it was assumed that one span of the bridge is 200 m (656 ft), width of one pier 12 m (39.3 ft), and effective width at the bridge axis is reduced by 25% of the total width of all piers. The results are shown in Table 7-5 and Fig. 7-8.

ii. Maximum water depth at constriction.

It is evident that water depth at thalweg is larger than mean depth and the larger the eccentricity of thalweg is, the larger the depth is. Hence deepness due to eccentricity of thalweg should be studied for design of guide-bank apron and bridge pier.

(i) Eccentricity of thalweg and the maximum water depth in the existing river.

When river width is denoted by B, distance between the center of the width and thalweg by E, eccentricity of the thalweg by 2E/B, mean water depth by R, and the depth at the thalweg by $H_{max}$, the relation between 2E/B and $H_{max}/R$ was studied with regard to some cross sections haveing remakable eccentricity which were selected from those surveyed by BWDB over the stretch from the confluence of the Ganges to the upstream 220 km from it and those surveyed by the Japanese team in 1973. The results are shown in Fig. 7-7.

In this figure, white circles are data obtained from cross sections surveyed by BWDB and black circles are those obtained from the surveying made by the Japanese team. Crosses show the results of study made by Lacey (114 GB). These data show no distinguishable difference between them, notwithstanding BWDB's were surveyed in the dry season and the Japanese team's

Table 7-4-1 Calculation of Equilibrium Depth for Existing River Channel

Nagarbari ( B = 5,200 m ; H = 10.218 m )

| Sect. No. | River width | | Mean depth hm | | Calculation of equilibrium depth | | | |
|---|---|---|---|---|---|---|---|---|
| | low water b (km) | total B (km) | ( m ) | ( ft ) | B / B₀ ( B/B ) | H/H₀ ( H/H ) | H ( m ) | H ( ft ) |
| N - 23 | 5.20 | 5.20 | 10.523 | 34.50 | 1.000 | 1.00 | 10.218 | 33.50 |
| 22 | 5.20 | 5.20 | 10.568 | 34.65 | 1.000 | 1.00 | 10.218 | 33.50 |
| 21 | 5.20 | 5.20 | 10.611 | 34.79 | 1.000 | 1.00 | 10.218 | 33.50 |
| 20 | 5.00 | 5.00 | 10.895 | 35.72 | 0.962 | 1.04 | 10.627 | 34.84 |
| 19 | 4.95 | 4.95 | 10.619 | 34.82 | 0.952 | 1.05 | 10.729 | 35.18 |
| 18 | 5.20 | 5.20 | 10.022 | 32.86 | 1.000 | 1.00 | 10.218 | 33.50 |
| 17 | 5.50 | 5.50 | 9.976 | 32.71 | (0.945) | (1.05) | 9.731 | 31.90 |
| 16 | 5.70 | 5.70 | 9.801 | 32.13 | (0.912) | (1.08) | 9.461 | 31.02 |
| 15 | 5.60 | 5.60 | 9.507 | 31.17 | (0.929) | (1.06) | 9.640 | 31.61 |
| 14 | 5.28 | 5.28 | 9.483 | 31.09 | (0.985) | (1.02) | 10.018 | 32.85 |
| 13 | 5.15 | 5.15 | 9.527 | 31.24 | 0.990 | 1.01 | 10.320 | 33.84 |
| 12 | 5.10 | 5.10 | 9.627 | 31.56 | 0.981 | 1.02 | 10.422 | 34.17 |
| 11 | 5.15 | 5.15 | 9.992 | 32.76 | 0.990 | 1.01 | 10.320 | 33.84 |
| 10 | 5.05 | 5.05 | 11.261 | 36.92 | 0.971 | 1.03 | 10.525 | 34.51 |
| 9 | 5.00 | 5.00 | 11.270 | 36.95 | 0.962 | 1.04 | 10.627 | 34.84 |
| 8 | 5.10 | 5.10 | 10.808 | 35.44 | 0.981 | 1.02 | 10.422 | 34.17 |
| 7 | 5.24 | 5.24 | 9.954 | 32.64 | (0.992) | (1.01) | 10.117 | 33.17 |
| 6 | 5.13 | 5.13 | 10.465 | 34.31 | 0.987 | 1.02 | 10.422 | 34.17 |
| 5 | 3.52 | 4.83 | 9.943 | 32.60 | 0.929 | 1.07 | 10.933 | 35.85 |
| 4 | 4.18 | 5.25 | 9.272 | 30.40 | (0.990) | (1.01) | 10.117 | 33.17 |
| 3 | 5.26 | 5.26 | 10.786 | 35.36 | (0.989) | (1.01) | 10.117 | 33.17 |
| 2 | 4.70 | 4.70 | 11.809 | 38.72 | 0.904 | 1.08 | 11.035 | 36.18 |
| 1 | 4.70 | 4.70 | 10.846 | 35.36 | 0.904 | 1.08 | 11.035 | 36.18 |

Sirajganj ( B = 6,350 m ; H = 8.561 m )

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| N- 20 | 6.35 | 6.35 | 8.815 | 27.92 | 1.000 | 1.00 | 8.561 | 28.07 |
| 19 | 6.28 | 6.28 | 9.675 | 30.74 | 0.989 | 1.01 | 8.647 | 28.35 |
| 18 | 6.06 | 6.06 | 9.590 | 30.46 | 0.954 | 1.03 | 8.818 | 28.91 |
| 17 | 6.05 | 6.05 | 9.525 | 30.25 | 0.953 | 1.03 | 8.818 | 28.91 |
| 16 | 6.00 | 6.35 | 8.988 | 28.49 | 1.000 | 1.00 | 8.561 | 28.07 |
| 15 | 5.23 | 6.09 | 8.784 | 27.82 | 0.959 | 1.03 | 8.818 | 28.91 |
| 14 | 4.18 | 5.58 | 8.843 | 38.01 | 0.879 | 1.09 | 9.331 | 30.59 |
| 13 | 4.50 | 5.35 | 9.374 | 29.75 | 0.843 | 1.13 | 9.674 | 31.72 |
| 12 | 3.84 | 4.80 | 9.948 | 31.63 | 0.756 | 1.21 | 10.359 | 33.96 |
| 11 | 4.14 | 4.50 | 10.610 | 33.80 | 0.709 | 1.27 | 10.872 | 35.65 |
| 10 | 3.59 | 4.10 | 11.032 | 35.19 | 0.646 | 1.35 | 11.557 | 37.89 |
| 9 | 3.30 | 4.05 | 11.409 | 36.42 | 0.638 | 1.36 | 11.643 | 38.17 |
| 8 | 3.40 | 4.50 | 9.951 | 31.64 | 0.709 | 1.27 | 10.872 | 35.65 |
| 7 | 4.30 | 4.80 | 10.253 | 32.63 | 0.756 | 1.21 | 10.359 | 33.96 |
| 6 | 5.00 | 5.00 | 9.532 | 30.27 | 0.787 | 1.18 | 10.102 | 33.12 |
| 5 | 5.75 | 5.75 | 9.099 | 28.85 | 0.906 | 1.07 | 9.160 | 30.03 |
| 4 | 6.01 | 6.01 | 9.175 | 29.10 | 0.946 | 1.04 | 8.903 | 29.19 |
| 3 | 6.46 | 6.46 | 9.990 | 31.77 | (0.983) | (1.01) | 8.476 | 27.79 |
| 2 | 5.99 | 6.50 | 8.881 | 28.13 | (0.977) | (1.02) | 8.393 | 27.52 |
| 1 | 5.10 | 6.50 | 8.717 | 27.60 | (0.977) | (1.02) | 8.393 | 27.52 |

Table 7 - 4 - 2 Calculation of Equilibrium Depth for Existing River Channel

Gabargaon ( $B_o$ = 8,500 m ; $H_o$ = 7.018 m )

| Sect. No. | River width | | Mean depth Hm | | Calculation of equilibrium depth | | | |
|---|---|---|---|---|---|---|---|---|
| | low water b (km) | total B (km) | ( m ) | ( ft ) | $B/B_o$ ($B_o/B$) | $H/H_o$ ($H_o/H$) | H ( m ) | H ( ft ) |
| G -14 | 7.23 | 8.50 | 6.663 | 21.85 | 0.897 | 1.00 | 7.018 | 23.01 |
| 13 | 6.00 | 9.00 | 6.647 | 21.79 | (0.944) | (1.05) | 6.684 | 21.91 |
| 12 | 6.03 | 8.50 | 7.060 | 23.15 | 1.000 | 1.00 | 7.018 | 23.01 |
| 11 | 5.22 | 7.00 | 7.530 | 24.69 | 0.824 | 1.15 | 8.071 | 26.46 |
| 10 | 4.91 | 5.50 | 8.776 | 28.77 | 0.647 | 1.35 | 9.474 | 31.06 |
| 9 | 4.86 | 5.15 | 9.799 | 32.13 | 0.606 | 1.42 | 9.966 | 32.68 |
| 8 | 5.50 | 5.50 | 8.870 | 29.08 | 0.647 | 1.35 | 9.474 | 31.06 |
| 7 | 5.90 | 5.90 | 8.250 | 27.05 | 0.694 | 1.29 | 9.053 | 29.68 |
| 6 | 6.67 | 6.67 | 8.138 | 26.68 | 0.785 | 1.19 | 8.351 | 27.38 |
| 5 | 5.99 | 7.00 | 7.436 | 24.38 | 0.824 | 1.15 | 8.071 | 26.46 |
| 4 | 6.00 | 7.65 | 7.005 | 22.97 | 0.900 | 1.09 | 7.650 | 25.08 |
| 3 | 7.45 | 8.50 | 7.032 | 23.06 | 1.000 | 1.00 | 7.018 | 23.01 |
| 2 | 7.45 | 8.50 | 7.143 | 23.42 | 1.000 | 1.00 | 7.018 | 23.01 |
| 1 | 7.45 | 8.50 | 7.235 | 23.72 | 1.000 | 1.00 | 7.018 | 23.01 |

Bahadurabad ( $B_o$ = 9,250 m ; $H_o$ = 6.123 m )

| Sect. No. | low water b (km) | total B (km) | ( m ) | ( ft ) | $B/B_o$ ($B_o/B$) | $H/H_o$ ($H_o/H$) | H ( m ) | H ( ft ) |
|---|---|---|---|---|---|---|---|---|
| B -14 | 7.82 | 9.25 | 5.344 | 17.52 | 1.000 | 1.00 | 6.123 | 20.08 |
| 13 | 7.82 | 10.00 | 5.330 | 17.48 | (0.925) | (1.07) | 5.722 | 18.76 |
| 12 | 6.13 | 9.50 | 5.684 | 18.64 | 0.974 | 1.02 | 6.245 | 20.48 |
| 11 | 6.00 | 7.00 | 7.604 | 24.93 | 0.757 | 1.21 | 7.409 | 24.29 |
| 10 | 4.80 | 6.00 | 8.192 | 26.86 | 0.649 | 1.35 | 8.266 | 27.10 |
| 9 | 4.06 | 5.65 | 7.514 | 24.64 | 0.611 | 1.41 | 8.633 | 28.30 |
| 8 | 4.45 | 5.00 | 8.578 | 28.12 | 0.541 | 1.53 | 9.368 | 30.71 |
| 7 | 4.70 | 4.70 | 9.238 | 30.29 | 0.508 | 1.60 | 9.797 | 32.12 |
| 6 | 5.35 | 5.35 | 9.381 | 30.76 | 0.578 | 1.47 | 9.001 | 29.51 |
| 5 | 6.45 | 6.45 | 7.823 | 25.65 | 0.697 | 1.30 | 7.960 | 26.10 |
| 4 | 7.57 | 7.57 | 7.340 | 24.07 | 0.818 | 1.15 | 7.103 | 23.29 |
| 3 | 8.05 | 8.05 | 6.916 | 22.68 | 0.870 | 1.10 | 6.735 | 22.08 |
| 2 | 8.65 | 8.65 | 6.872 | 22.53 | 0.935 | 1.06 | 6.490 | 21.28 |
| 1 | 9.00 | 9.00 | 6.822 | 22.37 | 0.973 | 1.02 | 6.245 | 20.48 |

Fig. 7.8-1 Longitudinal Profile Nagarbari site

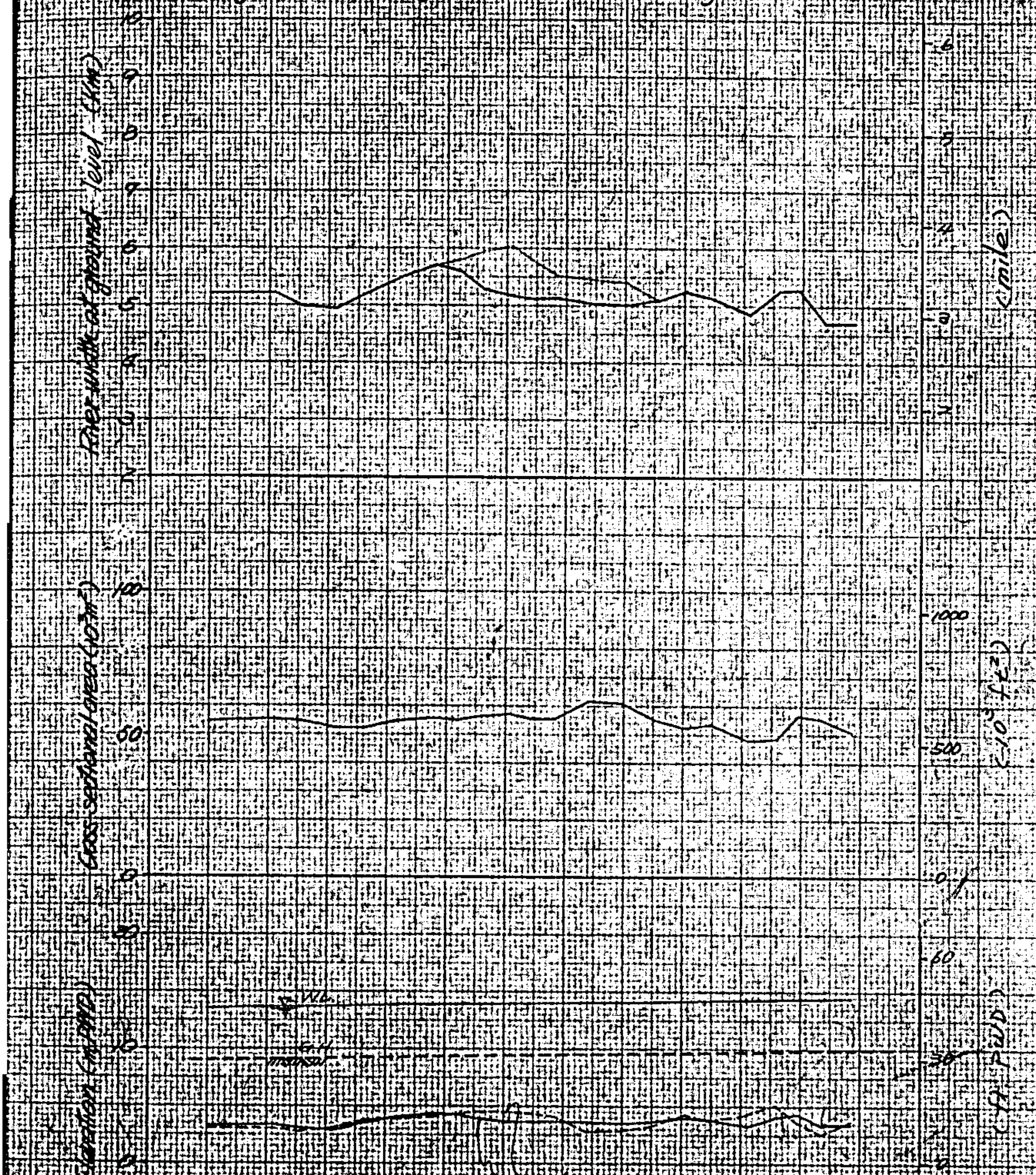


Fig-7-6-2 Longitudinal Profile : Sirajganj-site

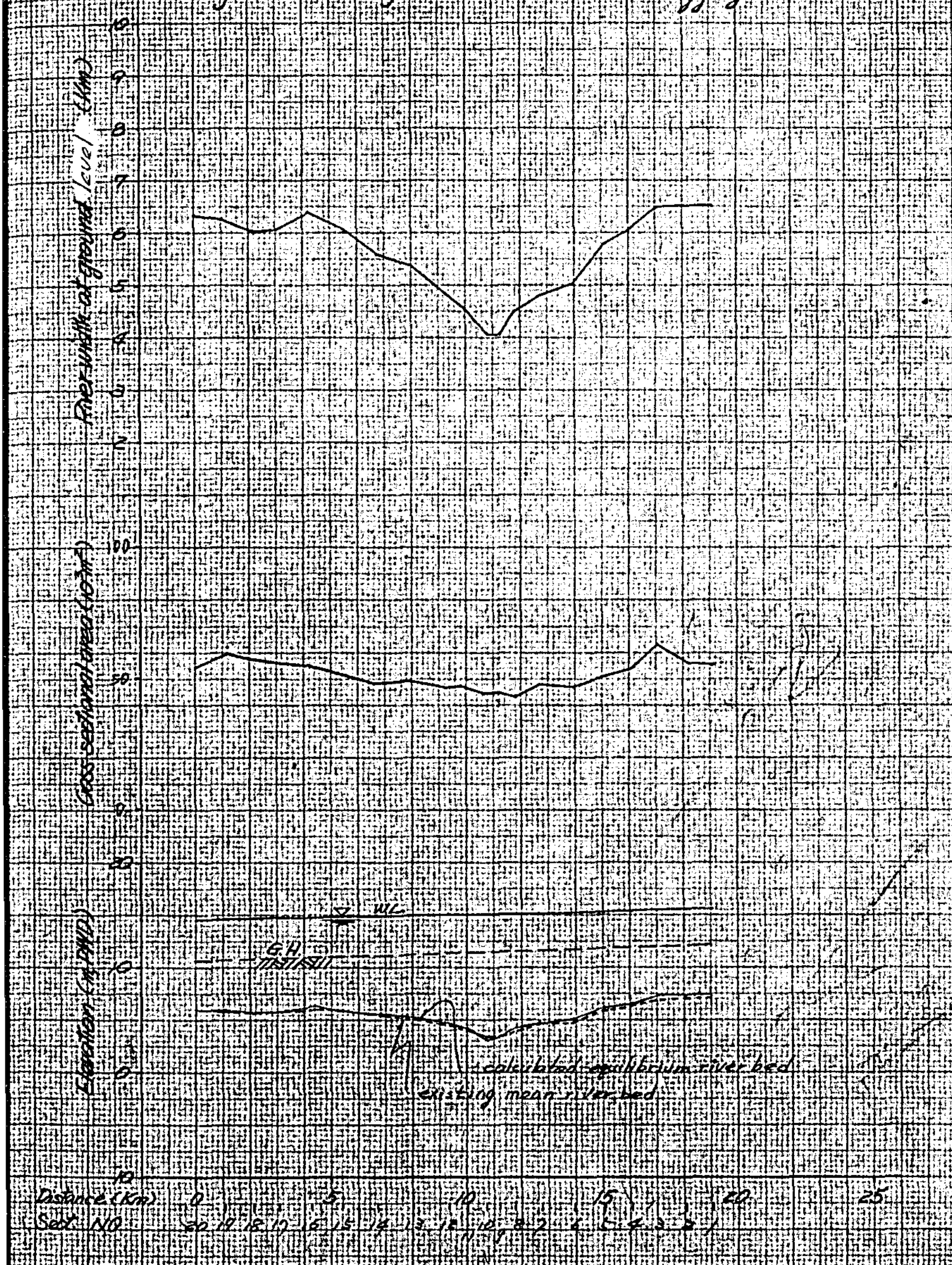


Fig. 7.6.3 Longitudinal Profile - Gabargaon site

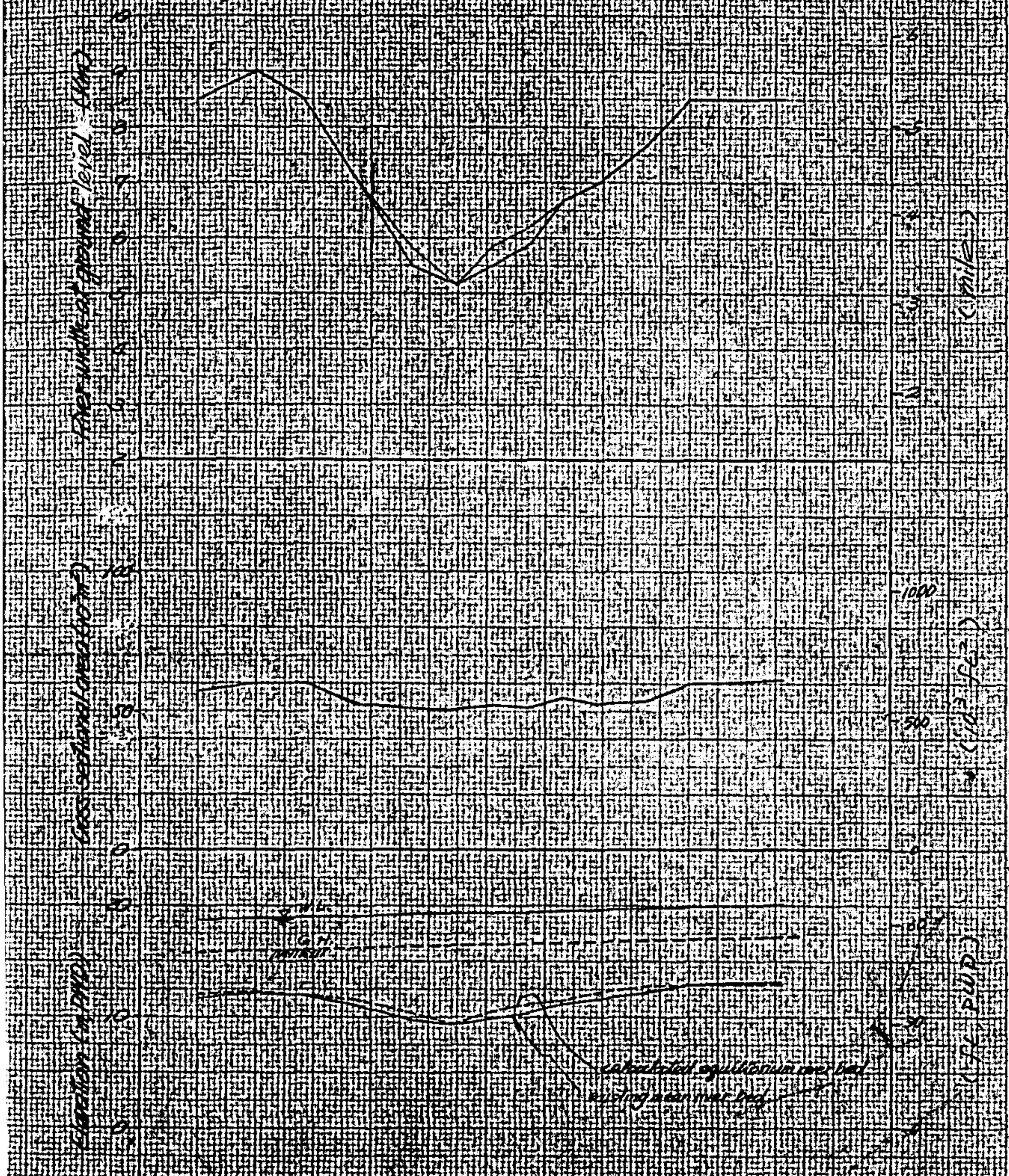
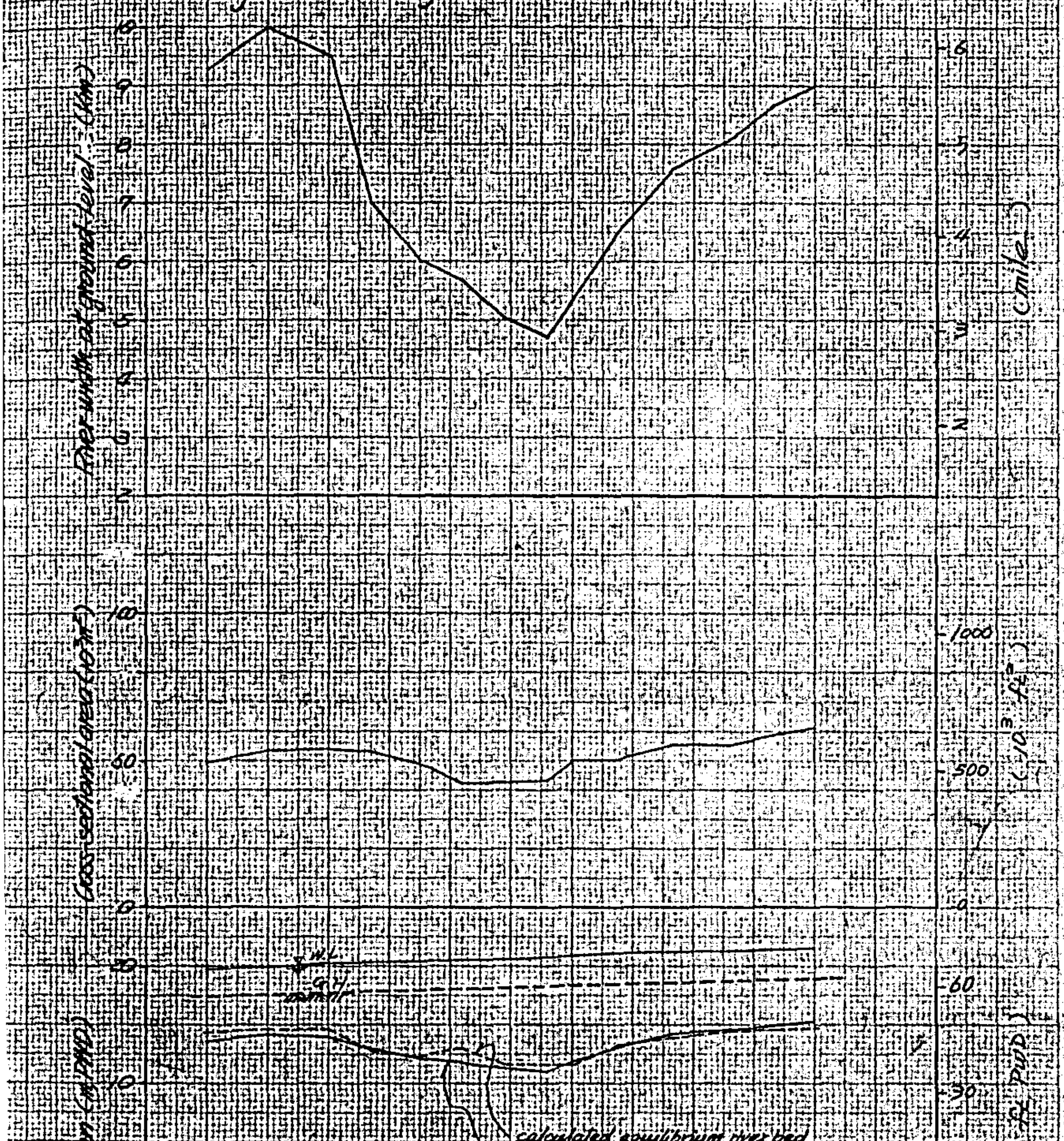


Fig. 7.6.4 Longitudinal Profile - Bahadurabad site

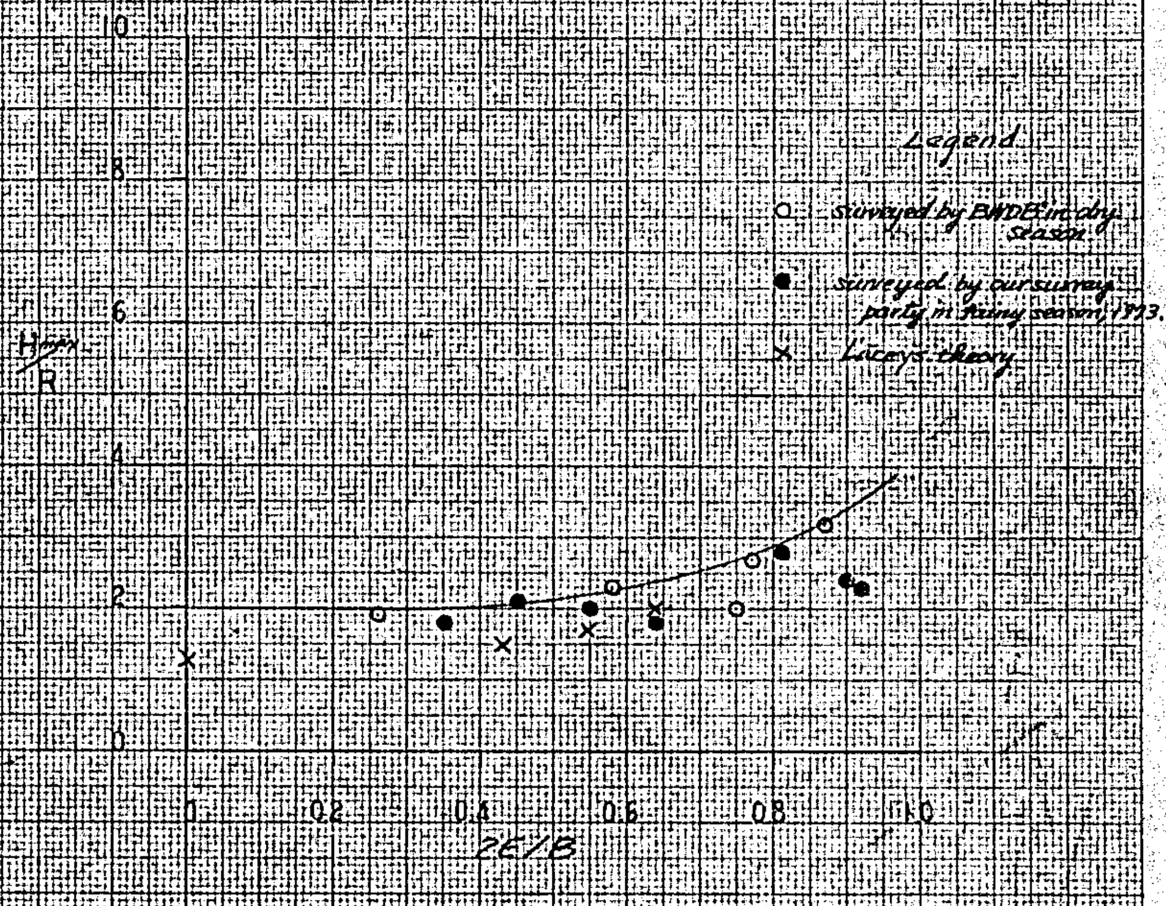



A4 180×250mm

Table 7 - 5 Calculation of Equilibrium Depth at the Constriction due to Guide Banks

| Site | type | Width between guide banks B (m) | Number of piers | Net width Be (m) | Calculation of equilibrium depth (B/B₀) | (H/H₀) | H (m (ft)) | Maximum depth (m (ft)) Hmax. at head of G.B. | Hmax at body of G.B. |
|---|---|---|---|---|---|---|---|---|---|
| Nagarbari $B_o$ = 5,200 m $H_o$ = 10.218 m | A | 2,000 | 9 | 1,865 | 0.359 | 2.03 | 20.743 (68.02) | 70.526 (231.26) | 56.139 (184.08) |
| | B | 4,200 | 20 | 3,900 | 0.750 | 1.22 | 12.466 (40.88) | 42.384 (138.98) | 42.384 (138.98) |
| | C | 5,200 | 25 | 4,825 | 0.928 | 1.06 | 10.831 (35.51) | 36.825 (120.75) | 36.825 (120.75) |
| Sirajganj $B_o$ = 6,350 m $H_o$ = 8.561 m | A | 2,000 | 9 | 1,865 | 0.294 | 2.33 | 19.949 (65.41) | 67.820 (222.36) | 53.862 (176.60) |
| | B | 4,200 | 20 | 3,900 | 0.614 | 1.40 | 11.985 (39.30) | 40.749 (133.60) | 40.749 (133.60) |
| | C | 5,600 | 27 | 5,195 | 0.818 | 1.15 | 9.845 (32.28) | 33.473 (109.75) | 33.473 (109.75) |
| Gabargaon $B_o$ = 8,500 m $H_o$ = 7.018 m | A | 2,000 | 9 | 1,865 | 0.219 | 2.89 | 20.282 (66.50) | 68.959 (226.12) | 54.891 (179.99) |
| | B | 4,200 | 20 | 3,900 | 0.459 | 1.72 | 12.071 (39.58) | 41.041 (134.57) | 41.041 (134.57) |
| | C | 5,200 | 25 | 4,825 | 0.568 | 1.49 | 10.457 (34.29) | 35.554 (116.58) | 35.554 (116.58) |
| Bahadurabad $B_o$ = 9,250 m $H_o$ = 6.123 m | A | 2,000 | 9 | 1,865 | 0.202 | 3.05 | 18.675 (61.24) | 63.495 (208.20) | 50.542 (165.73) |
| | B | 4,200 | 20 | 3,900 | 0.422 | 1.81 | 11.083 (36.34) | 37.682 (123.56) | 37.682 (123.56) |
| | C | 5,600 | 27 | 5,195 | 0.562 | 1.50 | 9.185 (30.12) | 31.229 (102.40) | 31.229 (102.40) |

# Fig. 7-7 Relation of Hmax/R to 2E/B




Data on Max Depth in Existing River

Data by BWDB

| Section NO. | Hmax (m) | R (m) | Hmax/R | 2E/B |
|---|---|---|---|---|
| 65/66 J0 9 | 32.0 | 10.1 | 3.17 | 0.872 |
| 62/88 2-1 | 27.5 | 12.0 | 2.30 | 0.575 |
| 68/69 7 | 25.9 | 13.6 | 1.90 | 0.264 |
| 62/90 7 | 23.1 | 8.5 | 2.72 | 0.787 |
| 66/87 9 | 22.1 | 11.0 | 2.01 | 0.746 |

Data by our survey Team

| Section NO | Hmax (m) | R (m) | Hmax/R | 2E/B |
|---|---|---|---|---|
| S-18 | 23.1 | 8.3 | 2.78 | 0.812 |
| N-22 | 22.9 | 12.9 | 1.78 | 0.436 |
| S-9 | 22.9 | 10.7 | 2.14 | 0.808 |
| S-9 | 22.4 | 12.3 | 1.82 | 0.350 |
| S-1 | 21.2 | 10.8 | 1.96 | 0.552 |
| G-4 | 20.8 | 8.6 | 2.37 | 0.936 |
| B-12 | 18.8 | 8.2 | 2.29 | 0.916 |

in the flood season. It is obvious, however, that water depth of thalweg has a definite tendency to increase rapidly with degree of eccentricity.

(ii) Maximum water depth.

According to Fig. 7-7, there seems to exist a practical upper limit in the eccentricity of thalweg. Therefore, if we take a value of 0.9 as the upper limit of eccentricity, we obtain 3.4 for the maximum value for it. The maximum depth for three types of construction at every bridge site was calculated using the value $H_{max}/R = 3.4$. The results are shown in Figs. 7-8-1 to 4.

In these figures, crosses show mean water depth below the design high water level and black circles show the maximum water depth for $H_{max}/R = 3.4$. White circle in Fig. 7-8-2 shows a maximum water depth in case a discharge, 2,500,000 cfs, corresponding to the design discharge for the Hardinge Bridge flowed through the constriction considered. White circle with cross in the figure shows the maximum water depth applied by Gales to the design of heads of guide banks in the Hardinge Bridge and triangle shows the depth applied to the bodys of guide banks. This figure indicates that the maximum water depth proposed by Gales well accords with that calculated here.

(iii) Design maximum water depth.

The maximum water depths for design of aprons was determined as follows.

a. Type-A.

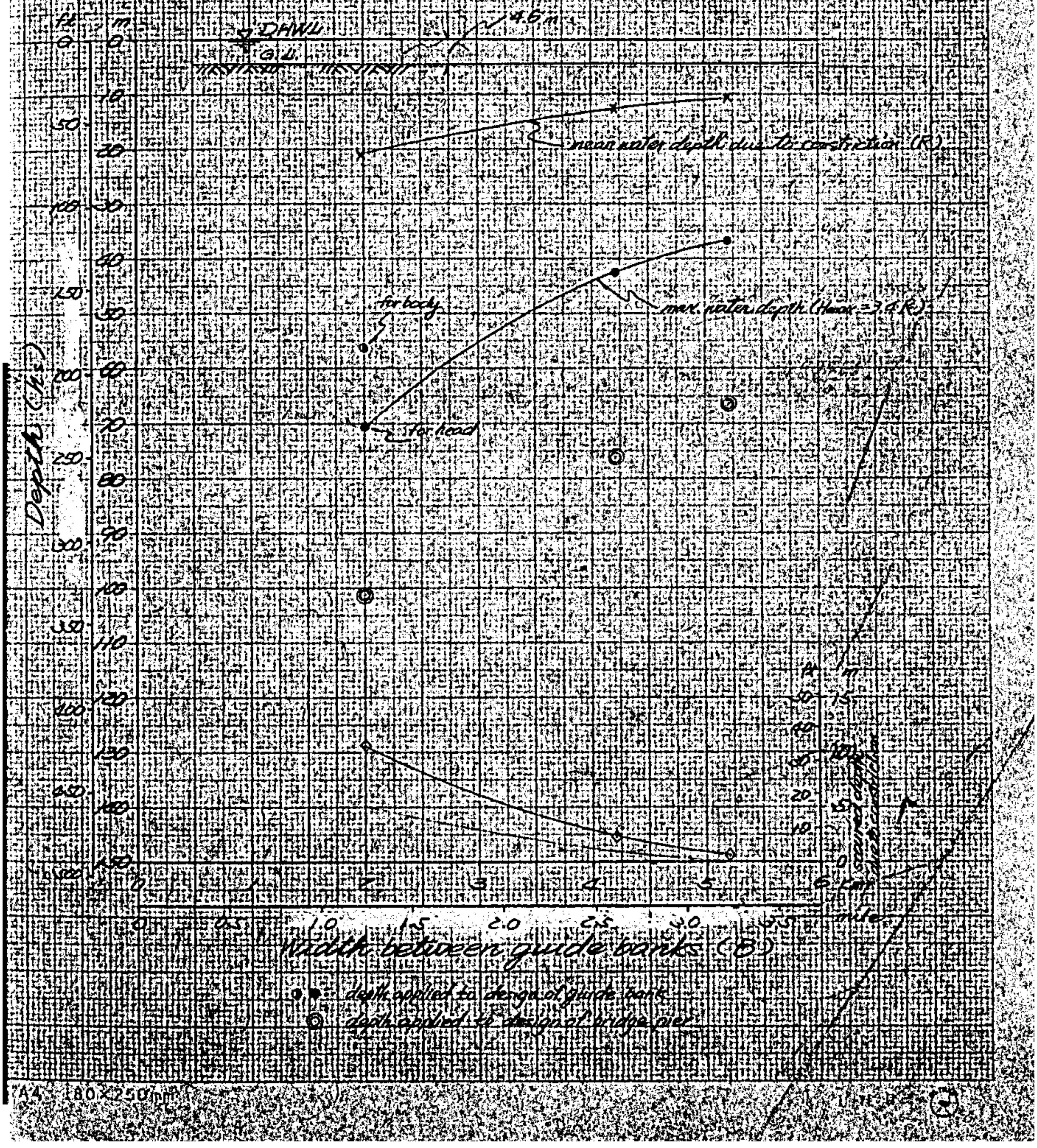
Head of guide bank : the maximum water depth calculated in the previous article.

71 m (233 ft) for Nagarbari site
68 m (223 ft) for Sirajganj site
69 m (226 ft) for Gabargaon site
64 m (210 ft) for Bahadurabad site.

Body and tail : design water depths for body and tail were determined by multiplying the above-mentioned design water depth for heads by Gales' ratio of water depth for body and tail to that of head.

57 m (187 ft) for Nagarbari site
54 m (177 ft) for Sirajganj site
55 m (180 ft) for Gabargaon site
51 m (167 ft) for Bahadurabad site.

Fig.7-8-1 Water Depth at Constriction due to Guide Bank
Nagarbari Site




A4 180×250mm

Fig 7-8-2 Water Depth at Constriction due to Guide Bank

Site: Sirajganj

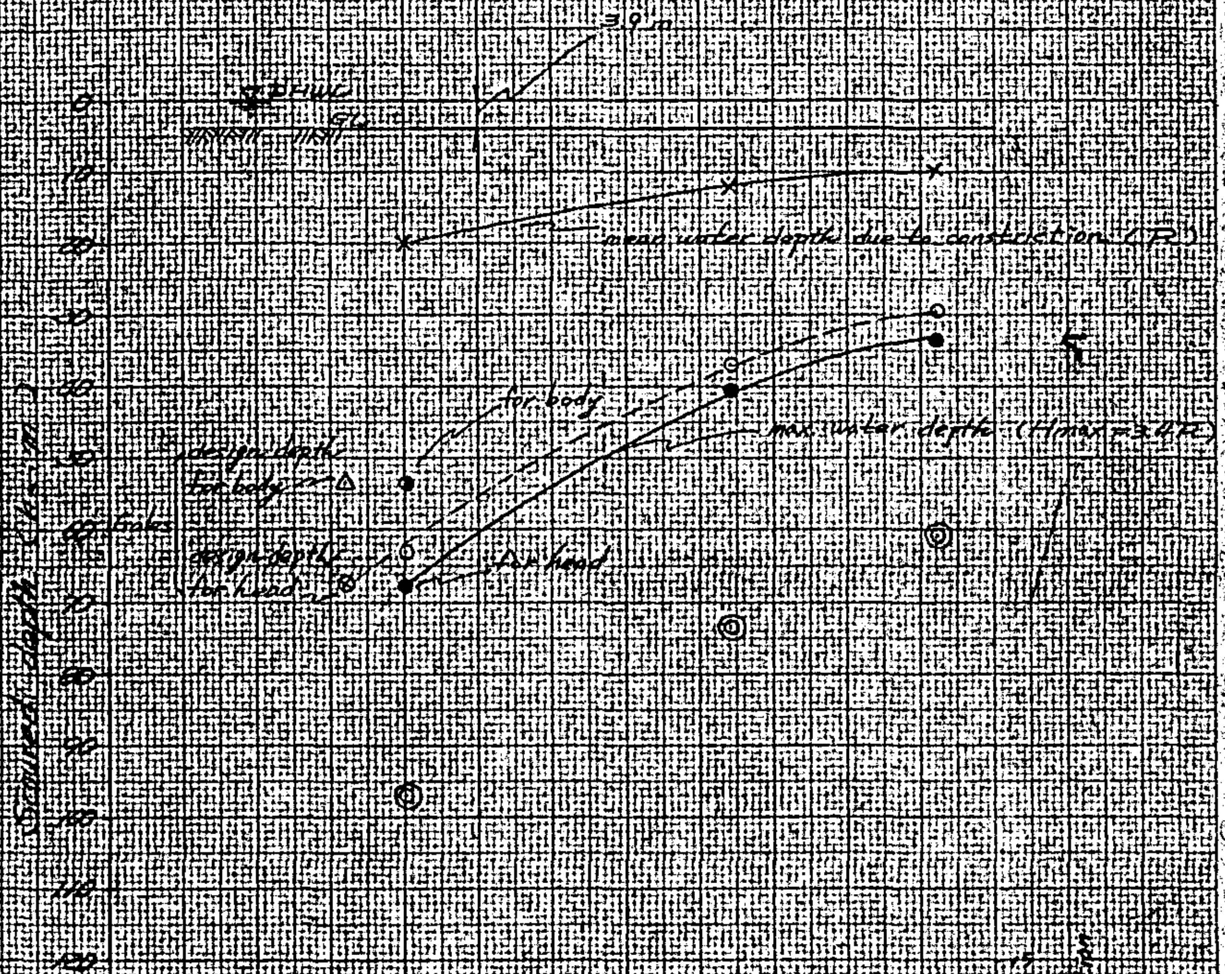


Fig. 7-8-3 Water Depth at Constriction due to Guide Bank
Gabargaon Site

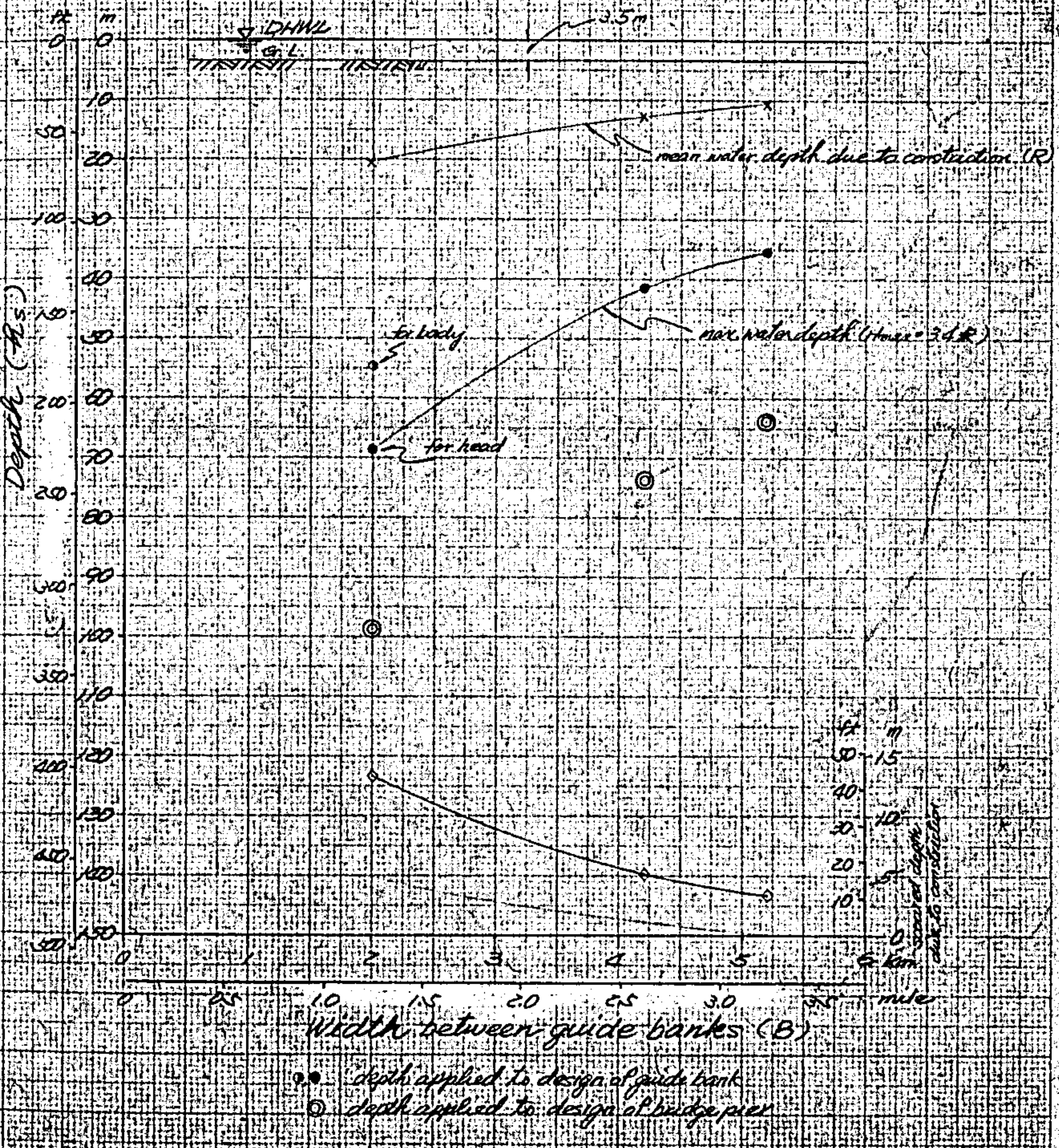
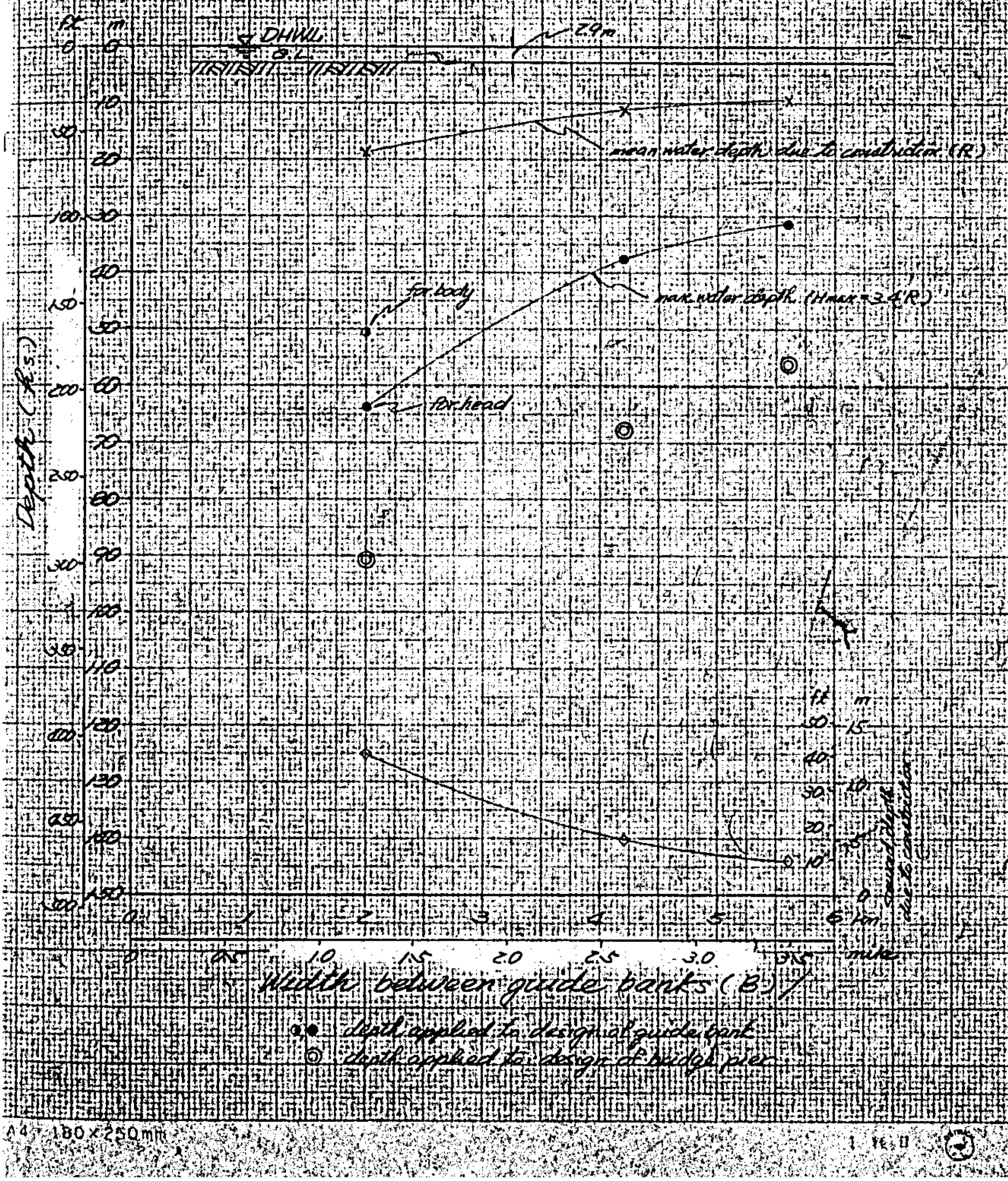


Fig. 7-8-4 Water Depth at Constriction due to Guide Bank

Bahadurabad Site



b. Type-B

It is difficult in this type to expect rectifying function of guide banks since width between them is quite large. Therefore, it was determined to adopt the same depth both for body and for tail as the depth calculated for head.

43 m (141 ft) for Nagarbari site
41 m (134 ft) for Sirajganj site
42 m (138 ft) for Gabargaon site
38 m (125 ft) for Bahadurabad site

c. Type-C.

On the basis of the same idea as in the case of Type-B, the following values were adopted for design.

37 m (121 ft) for Nagarbari site
34 m (111 ft) for Sirajganj site
36 m (118 ft) for Gabargaon site
32 m (105 ft) for Bahadurabad site

iii. Protection works for guide banks.

It will be expected that riprap which were placed in front of guide banks will fall with scouring at the foot and eventually achieve the purpose of revetment by covering the front of guide bank in a stable state of slope 1:2. For this purpose, an overall apron is shown in Fig. 7-9.

(i) Length of apron.

According to the figure, length of the apron is given as L = 1.5D where D is the water depth from the ground level at the apron to the prospective river bed.

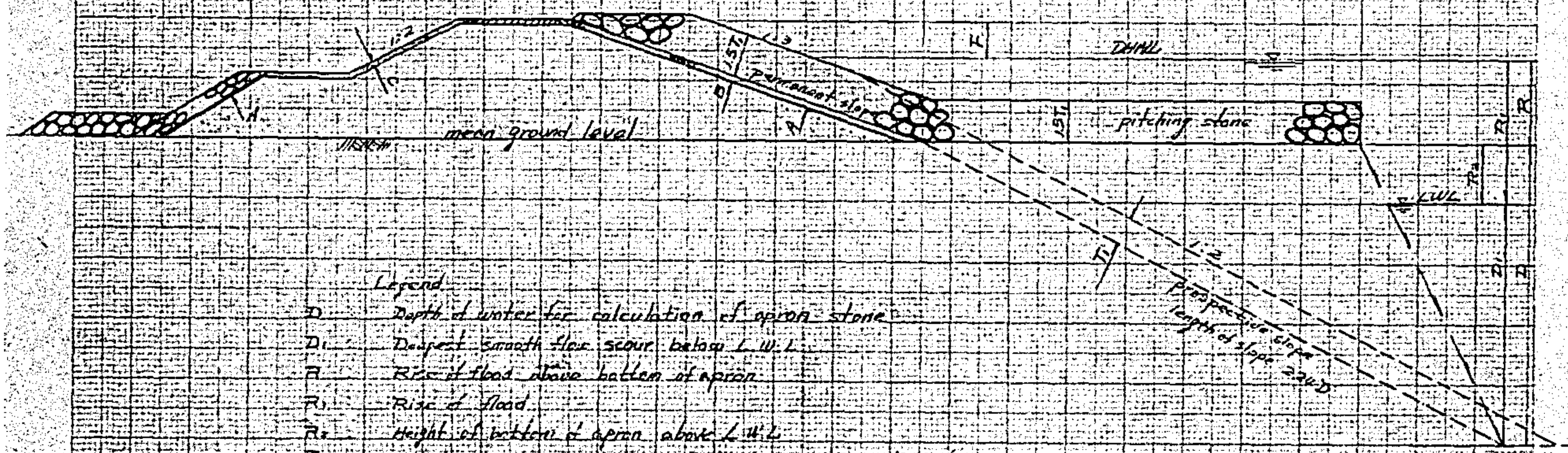
(ii) Thickness of pitching stones for apron.

It is necessary to provide a thickness enough to prevent the leakage of sand through the void of fallen apron stones. The thickness was taken at 7 ft for the heads of guide banks, 6 ft 3 in for the bodies and tails according to Gales' proposal. Let $T_1$ in the Fig. 7-9 be 7 ft and 6 ft 3 in, the required thickness T of the apron will be

$T = 1.5\ T_1 = 10$ ft 6 in for the beads
$T = 1.5\ T_1 = 9$ ft 3 in for the bodies and tails.

Fig. 7-9 Diagram of Overall Apron

Scale 1:50,000




Legend

D Depth of water for calculation of apron stone
D1 Deepest smooth flow scour below L.W.L
R Rise of flood above bottom of apron
R1 Rise of flood
R2 Height of bottom of apron above L.W.L
T1 Thickness of stone of prospective slope below bottom of apron
1.5T1 Thickness of stone of permanent slope and of apron
B Thickness of ballast (0.5m)
C Thickness of clay (0.5m)
F Free-board (2.0m)
A Thickness of asphalt mat

Considering an allowance, design thickness was taken as follows.

a. Type A.

3.5 m for heads

3.0 m for bodies and tails

b. Type B and C.

3.0 m for heads, bodies and tails

iv. Size of pitching stones.

Size of stones which do not move even at the prospective maximum depth was calculated for each type of guide banks for each of the proposed four sites. Kramer's formula was used for the critical tractive force.

$$u_*^2 = 26.95\ d \quad (c,g,s \text{ unit})$$

The results of calculation are shown in Table 7-6.

Table 7-6 Critical Grain Size Calculated by Kramer's Formula

| Site | Type | H (cm) | I | $u_* = \sqrt{gHI}$ (cm/s) | $u_*^2$ | d (cm) |
|---|---|---|---|---|---|---|
| Nagarbari | A, head | 7052.6 | 1/25400 | 16.496 | 272.118 | 10.097 |
| | A, body | 5613.9 | " | 14.717 | 216.590 | 8.037 |
| | B | 4238.4 | " | 12.788 | 163.533 | 6.068 |
| | C | 3682.5 | " | 11.920 | 142.086 | 5.272 |
| Sirajganj | A, head | 6782.0 | 1/18500 | 18.945 | 358.903 | 13.317 |
| | A, body | 5386.2 | " | 16.883 | 285.038 | 10.577 |
| | B | 4074.9 | " | 14.685 | 215.644 | 8.002 |
| | C | 3347.3 | " | 13.309 | 177.139 | 6.573 |
| Gabargaon | A, head | 6895.9 | 1/15000 | 21.226 | 450.543 | 16.718 |
| | A, body | 5489.1 | " | 18.937 | 358.610 | 13.306 |
| | B | 4104.1 | " | 16.375 | 268.141 | 9.950 |
| | C | 3555.4 | " | 15.241 | 232.288 | 8.619 |
| Bahadurabad | A, head | 6349.5 | 1/15000 | 20.367 | 414.815 | 15.392 |
| | A, body | 5054.2 | " | 18.172 | 330.222 | 12.253 |
| | B | 3768.2 | " | 15.690 | 246.176 | 9.135 |
| | C | 3122.9 | " | 14.284 | 204.033 | 7.571 |

These sizes are all within one-man stone called by Gales.

The sizes of stones to be pitched on the slope shall range from 60 to 100 kg since the weight of stones to resist to wave power was 52.2 kg as calculated in the previous Article (4).
(132 lb) (221 lb)

(6) Standard cross section of guide banks.

Standard cross sections of guide banks were designed on the basis of the studies mentioned in the previous Articles (1) to (5) and are shown in Fig. 7-10.

The lower diagram in Fig. 7-10 shows a cross section of a guide bank which will be constructed in the water. In this case, stone dikes will be first constructed at the both feet and then fine sand will be filled between them. The elevation of the top of the stone dike will be located near the mean water level and the apron will be placed on the existing ground surface at the foot of the dike.

Tables 7-7-1 to 7-7-3 show the prospective scoured depth D, the length of apron L and its thickness T in case guide banks are constructed on the mean ground level.

5. Design of Guide Banks and Closing Dikes.

The guide banks and the closing dikes were designed on the basis of the studies and the standard cross sections mentioned above. The alignments of these structures are shown in Figs. 7-11-1 to 7-11-4. Materials of the designed structures were estimated according to the following.

a. Guide bank.

Quantity of stones of stone dikes and a part of apron of 20 m width (66 ft) from the toe of the bank is contained in the quantity of pitching stones of the guide banks.

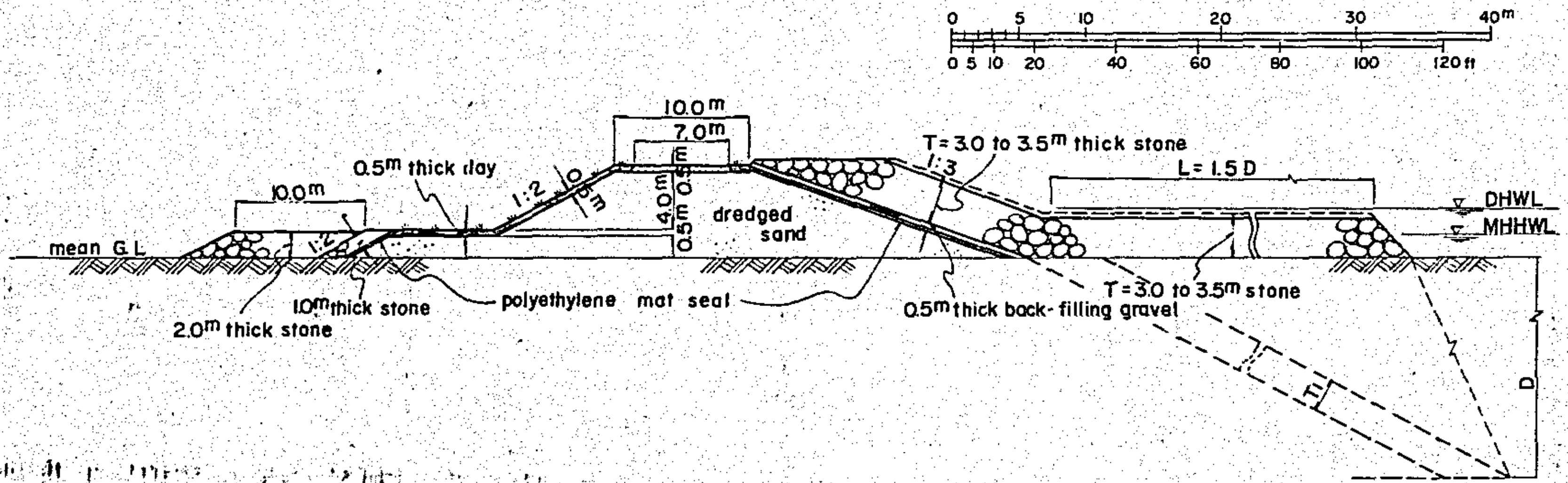
b. Apron.

Quantity of pitching stones for 20 m width of the apron was excluded from that of the apron.

c. Closing works.

In case of Type A and B, closing works were designed between the river banks and guide banks in order to facilitate the construction works of closing dikes. These are shown in Figs. 7-11-1 to 7-11-4 and the standard cross section is shown in Fig. 7-12.

Fig. 7-10 Standard Cross Section of Guide Bank (Scale 1/500)




( In case of deep water )

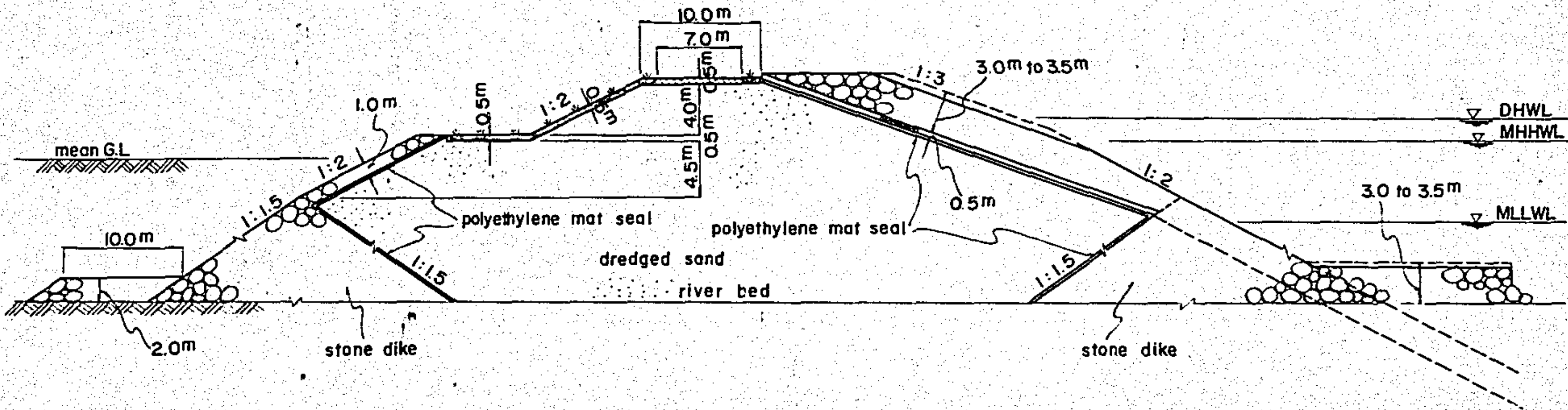


Table 7 - 7 - 1 Design Max. Scoured Depth : D
( Under Average Ground Level )

| Site | Part of G.B. | D | | | | | |
|---|---|---|---|---|---|---|---|
| | | Type | A | Type | B | Type | C |
| Nagarbari | Head | 66 m | (216.4) ft | 38 m | (124.6) ft | 33 m | (108.2) ft |
| | Body, Tail | 52 | (170.5) | 38 | (124.6) | 33 | (108.2) |
| Sirajganj | Head | 64 | (209.8) | 37 | (121.3) | 30 | ( 98.4) |
| | Body, Tail | 50 | (163.9) | 37 | (121.3) | 30 | ( 98.4) |
| Cabargaon | Head | 66 | (216.4) | 38 | (124.6) | 33 | (108.2) |
| | Body, Tail | 52 | (170.5) | 38 | (124.6) | 33 | (108.2) |
| Bahadurabad | Head | 61 | (200.0) | 35 | (114.8) | 29 | ( 95.1) |
| | Body, Tail | 48 | (157.4) | 35 | (114.8) | 29 | ( 95.1) |

Table 7 - 7 - 2 Design Length of Apron : L = 1.5 D

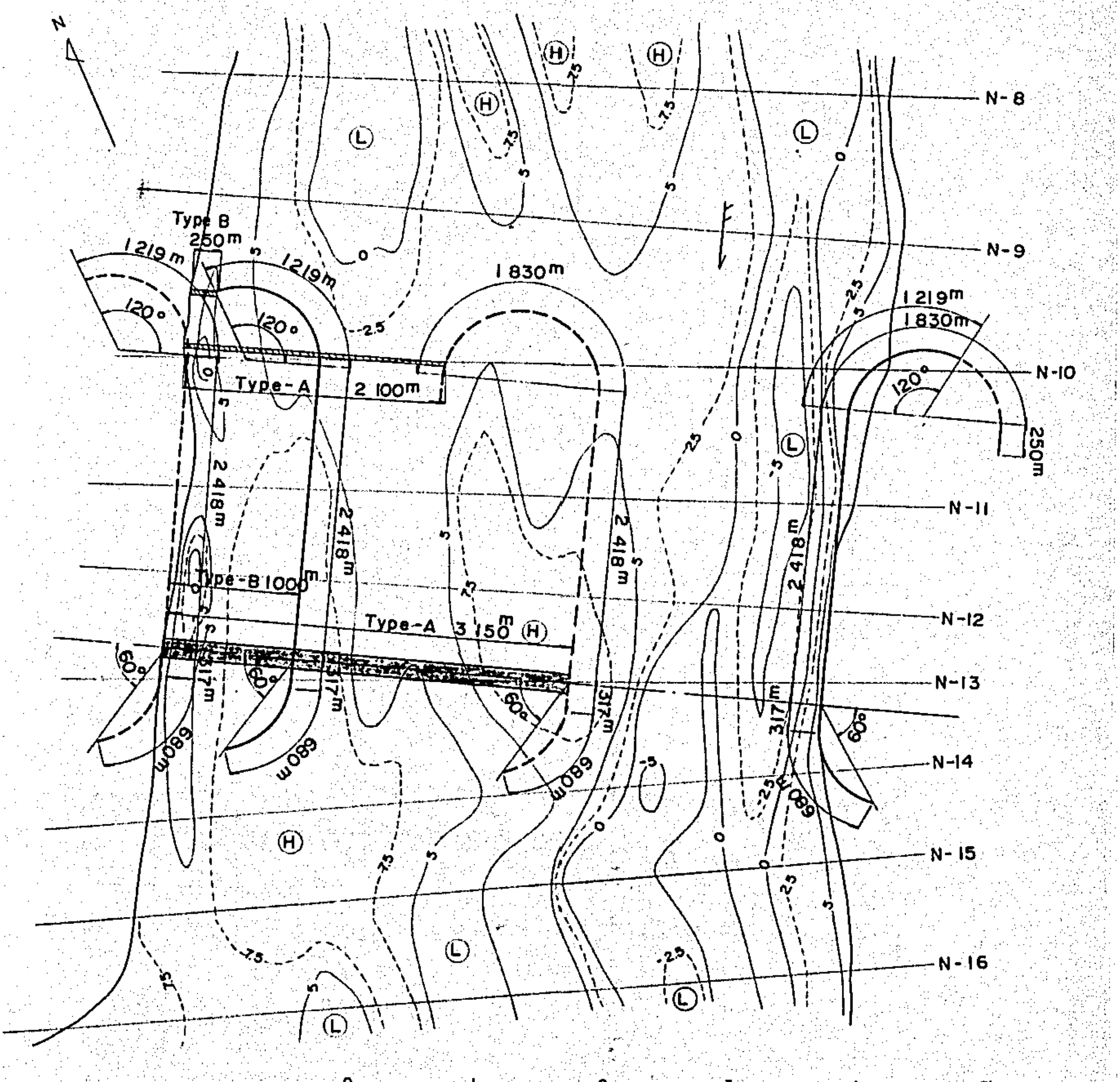
| Site | Part of G.B. | 1.5 D | | | | | |
|---|---|---|---|---|---|---|---|
| | | Type | A | Type | B | Type | C |
| Nagarbari | Head | 99 | (324.6) | 57 | (186.9) | 49.5 | (162.3) |
| | Body, Tail | 78 | (255.7) | 57 | (186.9) | 49.5 | (162.3) |
| Sirajganj | Head | 96 | (314.8) | 55.5 | (182.0) | 45 | (147.5) |
| | Body, Tail | 75 | (245.9) | 55.5 | (182.0) | 45 | (147.5) |
| Cabargaon | Head | 99 | (324.6) | 57 | (186.9) | 49.5 | (162.3) |
| | Body, Tail | 78 | (255.7) | 57 | (186.9) | 49.5 | (162.3) |
| Bahadurabad | Head | 91.5 | (300.0) | 52.5 | (172.1) | 43.5 | (142.6) |
| | Body, Tail | 72 | (236.1) | 52.5 | (172.1) | 43.5 | (142.6) |

Table 7 - 7 - 3 Thickness of Stone of Slope, Apron(T)

| Type | Permanent slope | | Apron | | Prospective slope | |
|---|---|---|---|---|---|---|
| | Head | Body, Tail | Head | Body, Tail | Head | Body, Tail |
| A | 3.5(11.5) | 3(9.8) | 3.5(11.5) | 3(9.8) | 2.34(7.7) | 2(6.6) |
| B | 3 ( 9.8) | 3(9.8) | 3 ( 9.8) | 3(9.8) | 2 (6.6) | 2(6.6) |
| C | 3 ( 9.8) | 3(9.8) | 3 ( 9.8) | 3(9.8) | 2 (6.6) | 2(6.6) |

# Fig. 7-11-1 Location of Guide Bank, Closing Dike and Closing Works

SITE : Nagarbari




SCALE : 1 / 50,000

LEGEND :

- ——— : Type - A ⎫
- ——— : Type - B ⎬ Guide bank
- - - - : Type - C ⎭
- : Closing dike
- : Closing work
- —·— : Bridge axis

DIMENSION :

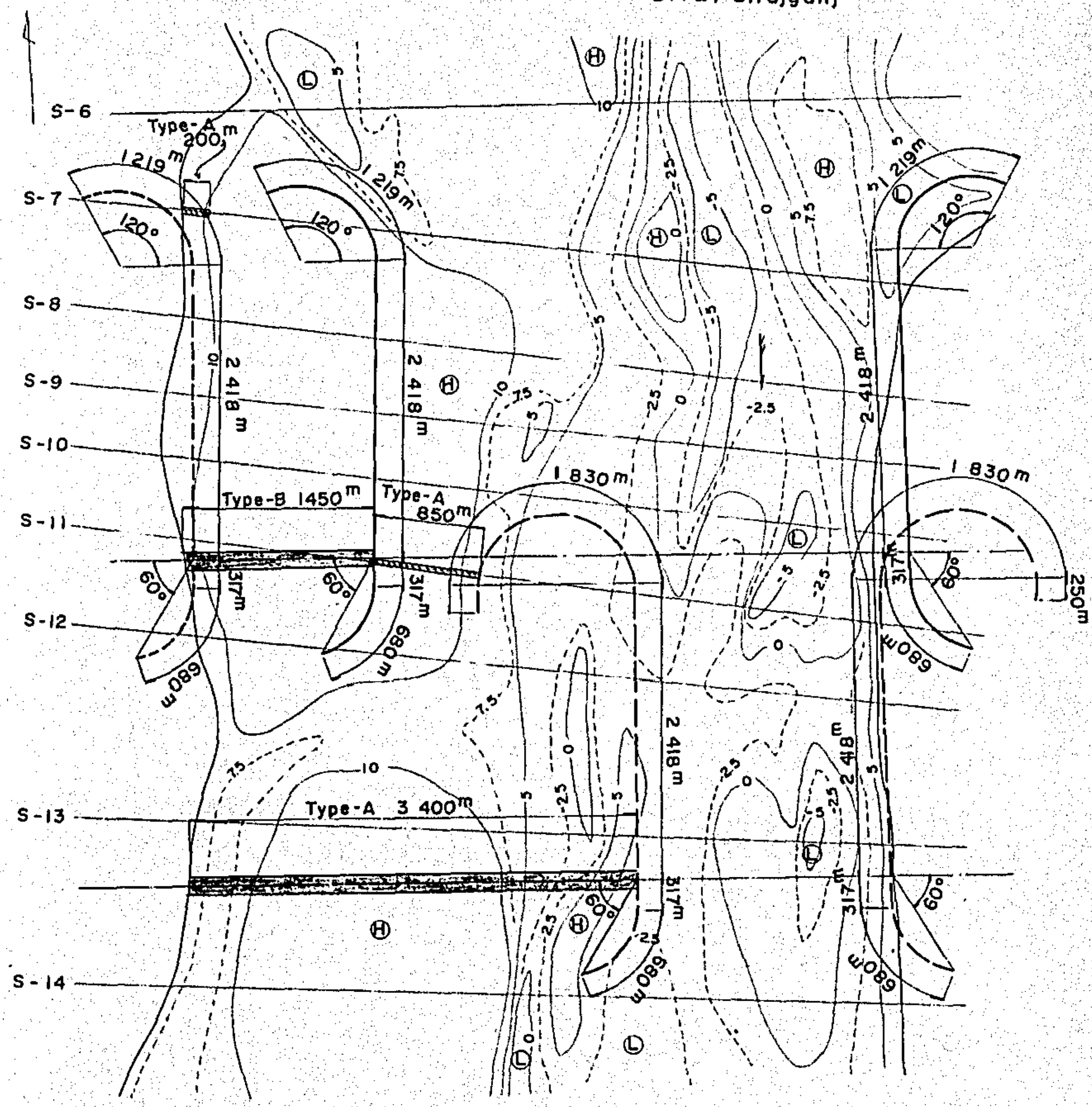
( unit : m )

| TYPE | Guide bank | | Closing dike | | Closing work | |
|---|---|---|---|---|---|---|
| | Channel width | Length | Right side | Left side | Right side | Left side |
| A | 2 000 | 5 495 x 2 | 3 150 | 0 | 2 100 | 0 |
| B | 4 200 | 4 634 x 2 | 1 000 | 0 | 250 | 0 |
| C | 5 200 | 4 634 x 2 | 0 | 0 | 0 | 0 |

CONTOUR LINE : in m, PWD.

# Fig.7-11-2 Location of Guide Bank, Closing Dike and Closing Works

SITE: Sirajganj




SCALE : 1 / 50,000

0 1 2 3 4 5km

0 0.5 1 2 3 mile

LEGEND :

- ——— : Type-A | Guide bank
- ——— : Type-B | Guide bank
- ------ : Type-C | Guide bank
- Closing dike
- Closing work
- Bridge axis

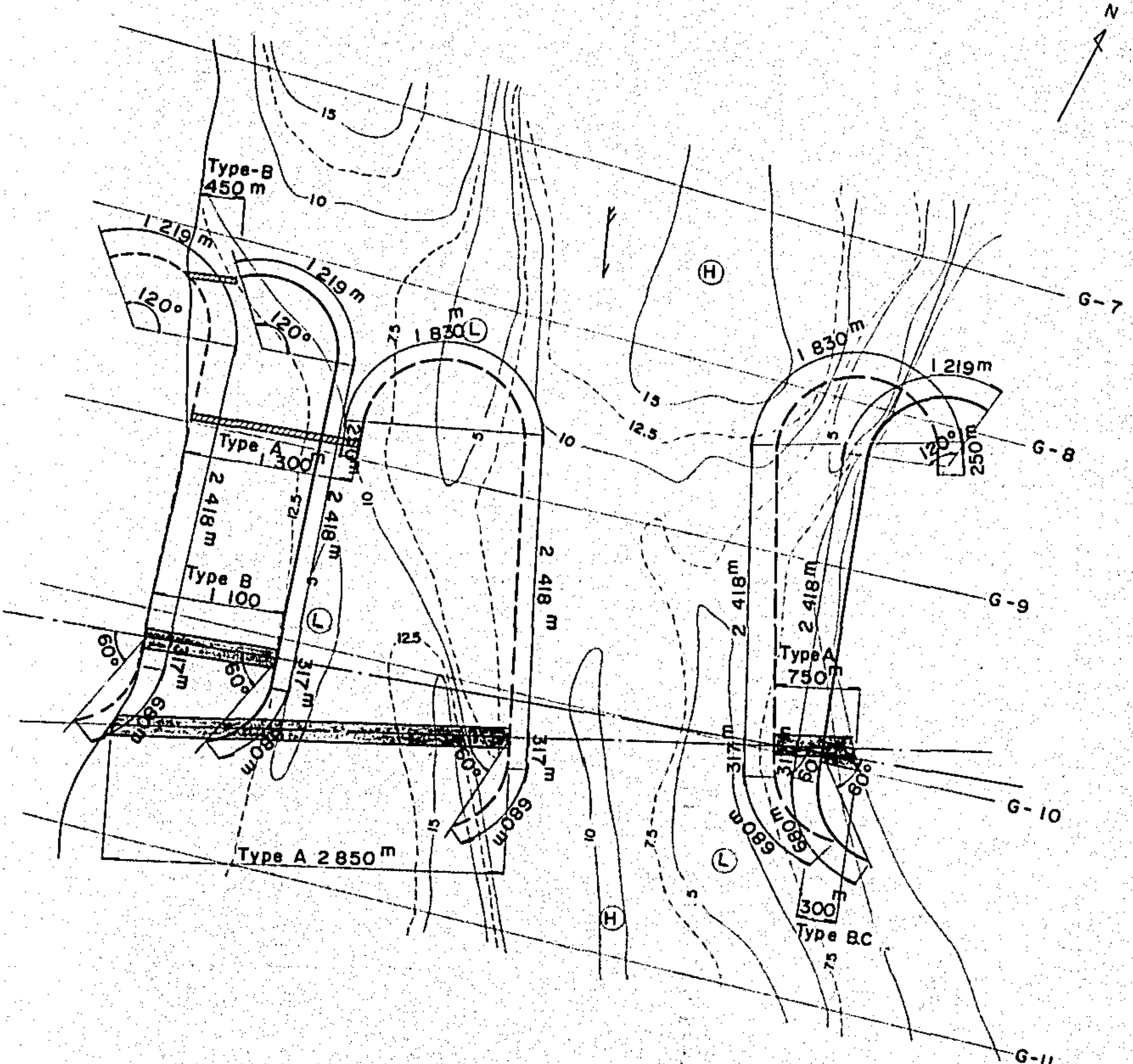
DIMENSION : ( unit : m )

| TYPE | Guide bank | | Closing dike | | Closing work | |
|---|---|---|---|---|---|---|
| | Channel width | Length | Right side | Left side | Right side | Left side |
| A | 2 000 | 5 495 x 2 | 3 400 | 0 | 1 050 | 0 |
| B | 4 200 | 4 634 x 2 | 1 450 | 0 | 200 | 0 |
| C | 5 600 | 4 634 x 2 | 0 | 0 | 0 | 0 |

CONTOUR LINE : in m, PWD.

# Fig. 7-11-3 Location of Guide Bank, Closing Dike and Closing Works

SITE : Gabargaon




SCALE : 1/ 50,000

0 1 2 3 4 5 km

0 0.5 1 2 3 mile

LEGEND :

- — — — : Type - A
- ——— : Type - B
- - - - - : Type - C

Guide bank

- : Closing dike
- : Closing work
- — · — : Bridge axis

DIMENSION :

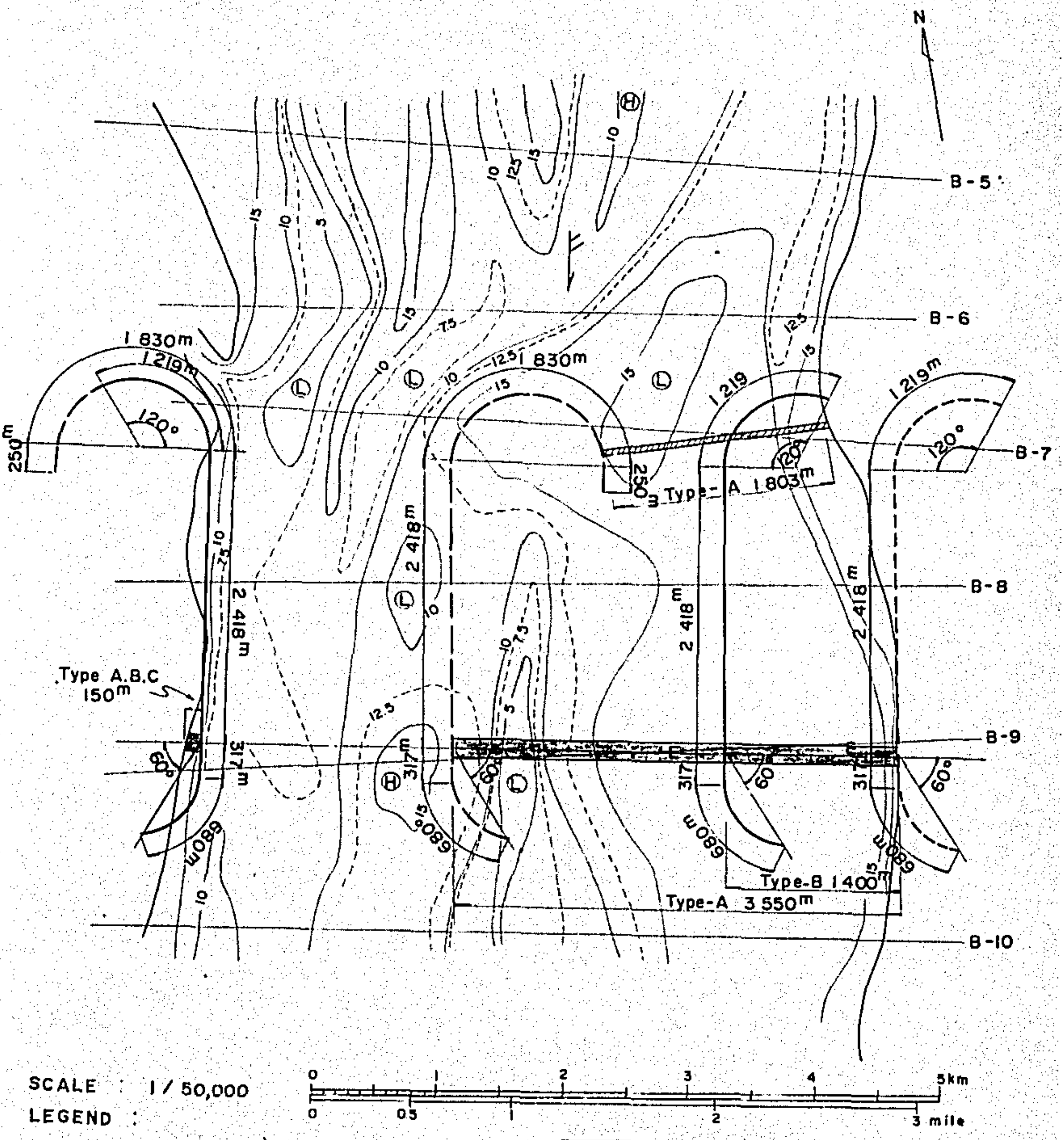
(unit : m)

| TYPE | Guide bank | | Closing dike | | Closing work | |
|---|---|---|---|---|---|---|
| | Channel width | Length | Right side | Left side | Right side | Left side |
| A | 2 000 | 5 495 x 2 | 2 850 | 750 | 1 300 | 0 |
| B | 4 200 | 4 634 x 2 | 1 100 | 300 | 450 | 0 |
| C | 5 200 | 4 634 x 2 | 0 | 300 | 0 | 0 |

CONTOUR LINE : In m, PWD.

# Fig.7-11-4 Location of Guide Bank, Closing Dike and Closing Works

SITE : Bahadurabad




SCALE : 1 / 50,000

LEGEND :

- — — — : Type – A ⎫
- ———— : Type – B ⎬ Guide bank
- - - - - : Type – C ⎭
- : Closing dike
- : Closing work
- —·— Bridge axis

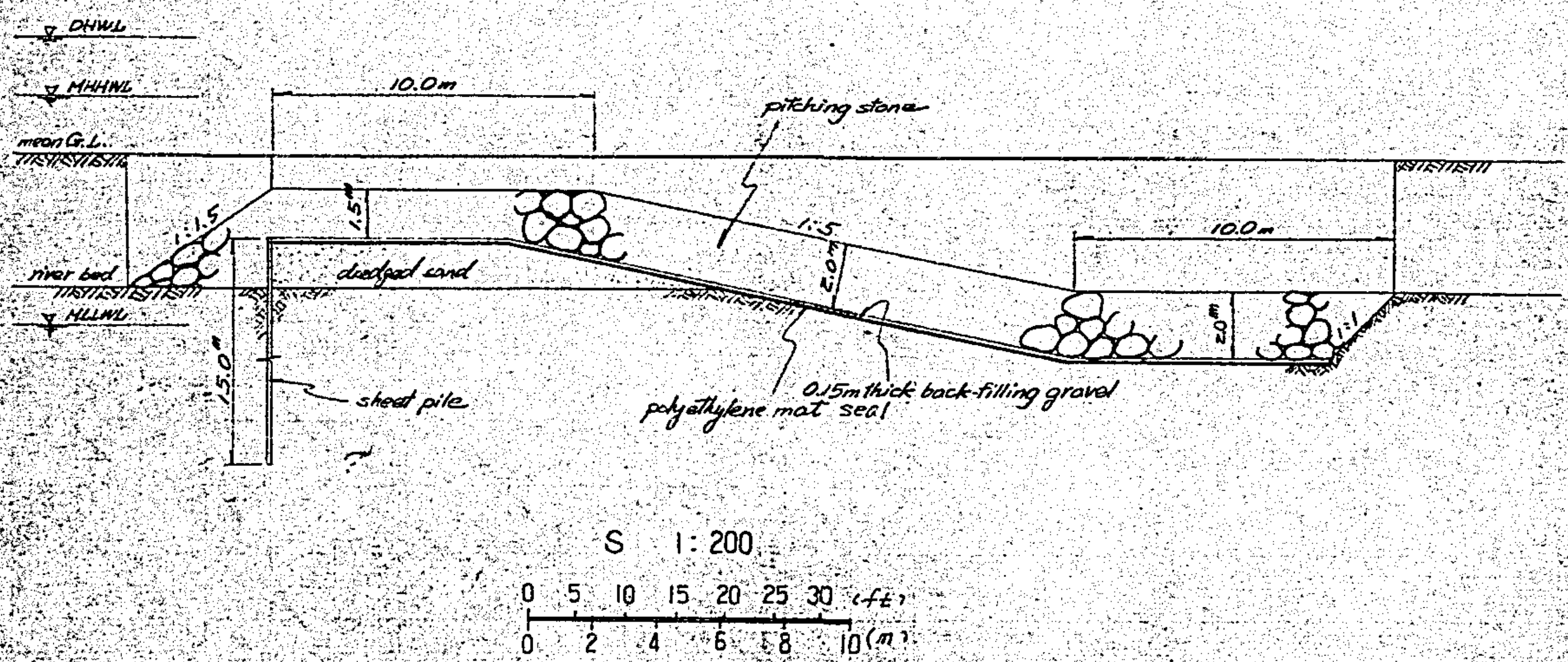
DIMENSION :

(unit : m )

| TYPE | Guide bank | | Closing dike | | Closing work | |
|---|---|---|---|---|---|---|
| | Channel Width | Length | Right side | Left side | Right side | Left side |
| A | 2 000 | 5 495 x 2 | 150 | 3 550 | 0 | 1 805 |
| B | 4 200 | 4 634x 2 | 150 | 1 400 | 0 | 0 |
| C | 5 600 | 4 634 x 2 | 150 | 0 | 0 | 0 |

CONTOUR LINE : in m , PWD.

Fig. 7-12 STANDARD CROSS SECTION OF CLOSING WORK



d. Closing dikes.

Closing dikes were designed to support the function of the guide banks and are shown in Figs. 7-11-1 to 7-11-4. The standard cross section is given in Fig. 7-13.

Fig. 7-13-1 Typical Cross Section of Approach Road (Scale 1/2,000)

(Double track railway & four lane road, in the vicinity of guide bank)

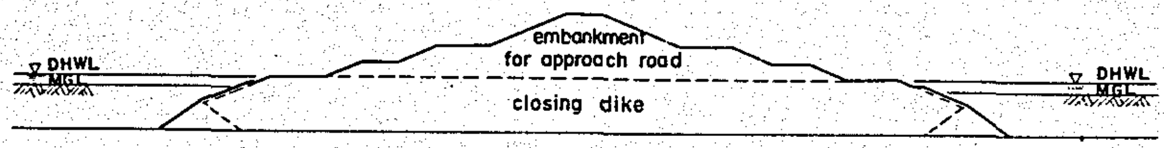
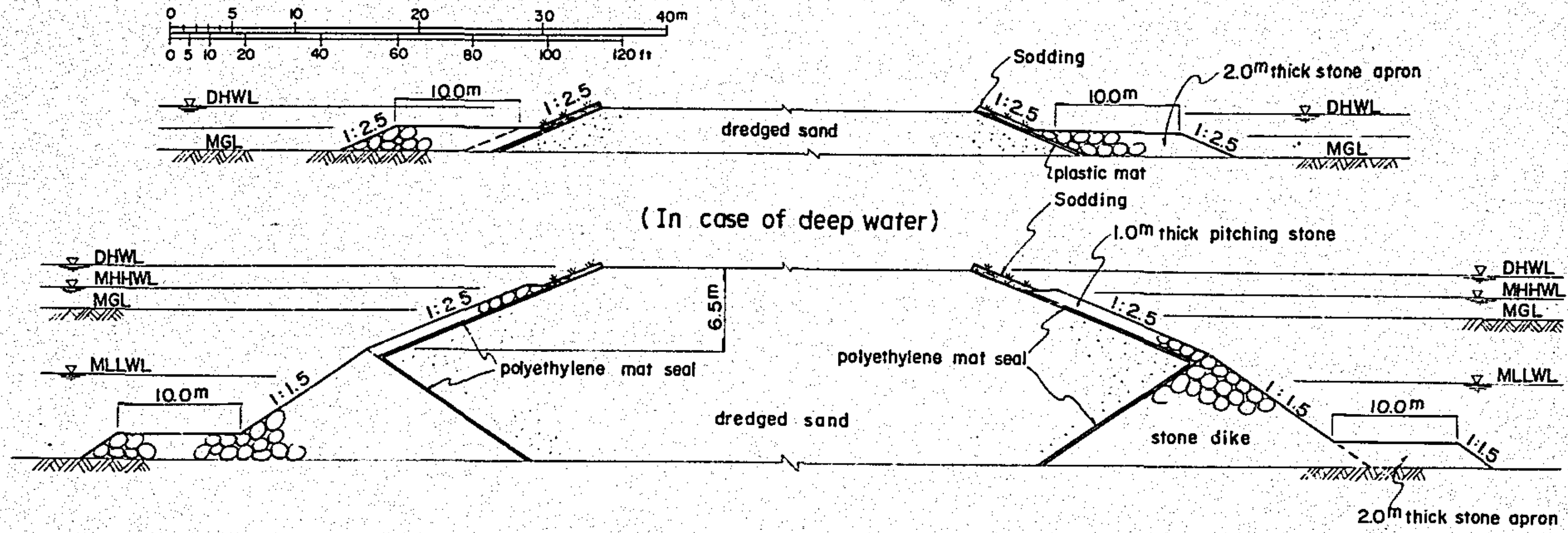



Fig. 7-13-2 Standard Cross Section of Closing Dike (Scale 1/500)



# CHAPTER VIII
# SCOUR AT BRIDGE PIERS

## 1. Studies on Depth of Scour at Bridge Piers.

P. Andru (124 GB) has obtained the following formula expressing the relation between scoured depth, bed factor $Q^2/B^2H^3$ and unit discharge on the basis of field measurements and model tests.

$$H_s(Q^2/B^2H^3)^{1/3} = 1.8(Q/B)^{2/3} \qquad (1)$$

or

$$H_s = 1.8\ H$$

where $H_s$ = scoured depth measured downwards from the water surface
$Q$ = discharge
$B$ = width of wide rectangular channel
$H$ = water depth.

On the other hand, E.M.Laursen has obtained the following formula on the basis of extensive experiments (123 GB).

$$H_s = H + d_s \times k_s \times k_\alpha \times k_\tau \qquad (2)$$

where $d_s$ = scoured depth measured downwards from the river bed before scour; a curve of H/b and $d_s$/b is prepared in Fig. 8-2-1,
$b$ = width of a pier,
$k_s$ = shape coefficient for nose forms to be used only for piers aligned with flow; Fig. 8-2-2 is prepared,
$k_\alpha$ = multiplying factor to be applied to the depth of scour obtained from the basic curve; a family of curves for $\alpha$, L/b and $k_\alpha$ is prepared in Fig. 8-2-3,
$\alpha$ = angle of attack between the pier and the flow,
$L$ = length of a pier in the direction of the flow,
$k_\tau$ = ratio of depth of scour under suspended-load conditions to depth of scour under bed-load conditions; curves for $d_s/H$, $u_*/w_o$ and $k_\tau$ are prepared in Fig. 8-2-4; $k_\tau = 1$ in the case of $u_*/w_o < 1/2$,
$u_*$ = friction velocity,
$w_o$ = settling velocity of sediment particle.

Fig. 8-1 Relation between Scored Depth and Discharge by Andru

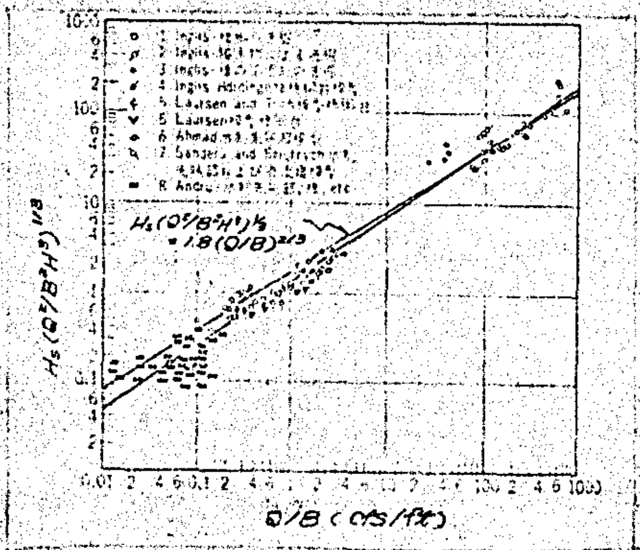



Fig. 8-2 Diagrams for the Calculation of Scoured Depth by Laursen

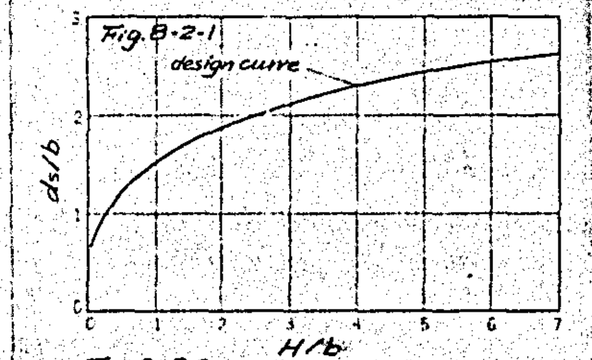
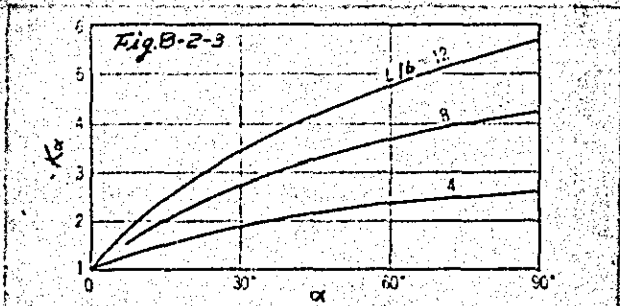



Fig. 8-2-2

| 頭部形状 | 長さと幅の比 | $K_s$ |
|---|---|---|
| 長方形 | | 1.00 |
| 半円形 | | 0.90 |
| 長円形 | 2 : 1 | 0.80 |
| | 3 : 1 | 0.75 |
| レンズ形 | 2 : 1 | 0.80 |
| | 3 : 1 | 0.70 |



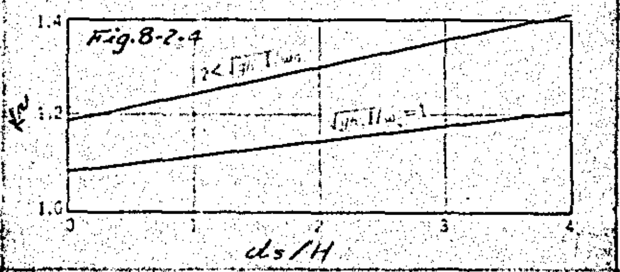
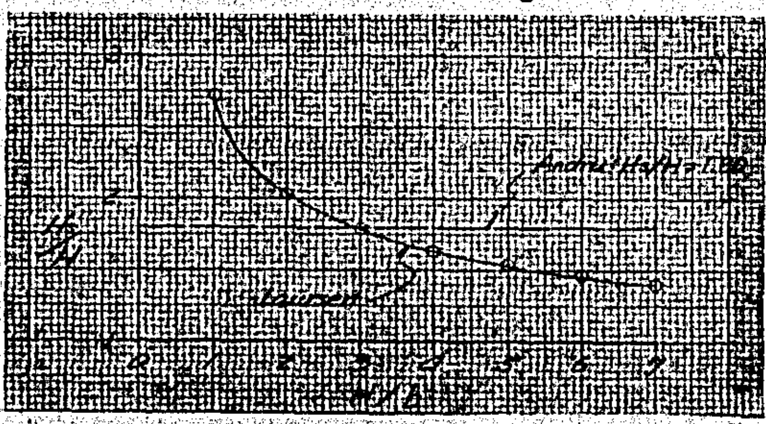
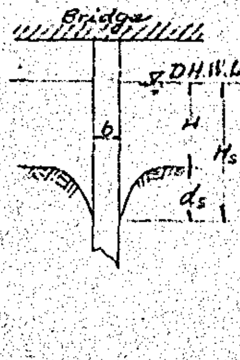




Fig. 8-4 Hs/H Curve for Jamuna Bridge



According to the study made by Breusers (133 GB), the equilibrium depth of scour with continuous sediment motion is given by

$$d_s = 1.4\ b \qquad (3)$$

for circular cylindrical piers. On the other hand, Larras (134 GB) found that

$$d_s = 1.05\ b^{0.75} \qquad (d_s, b:m) \qquad (4)$$

also for circular cylindrical piers.

Shen, Schneider and Karaki gave a formula

$$d_s = 0.000223\ R_e^{0.619} \qquad (d_s:m) \qquad (5)$$

for the equilibrium scour depth in the clear-water scour region. $R_e$ means the pier Reynolds number

$$R_e = Ub/\nu$$

where U is the mean velocity of the undisturbed flow and $\nu$ the Kinematic viscosity.

The above-mentioned formulas from (1) to (5) are shown in Fig. 8-3 in comparison. The values by Andru's, Laursen's and Shen's formulas are given with regard to the case of Sirajganj site.

As seen in this figure, these formulas give a very wide range of values. It can be said, however, that (1) Andru's formla cannot hold with regard to piers of small diameter since scoured depth depends on water depth alone, (2) Laursen's formula may hold with regard to piers of larger diameter because the formula was obtained for the range of H/b = 1 to 7, (3) Breuser's and Larras' formulas seem to be unreasonable at least in case of larger diameter because these formulas are independent of water velocity and bed material and relate to pier width alone, but they seem to hold with regard to piers of small diameter such as pipes, and (4) Shen's formula seems to hold over a considerably wide range of pier width.

After all, in our judgement, Andru's formula is applicable for a pier width larger than about 10 m and Shen's formula for a pier width smaller than about 3 m in such case of large depth as in the Jamuna River.

2. Scour Depth around Wider Piers.

If we adopt a well type as substructure, the width or the diameter will be of the order of 12 m. If we assume that b = 12 m, $k_s$ = 0.90 and

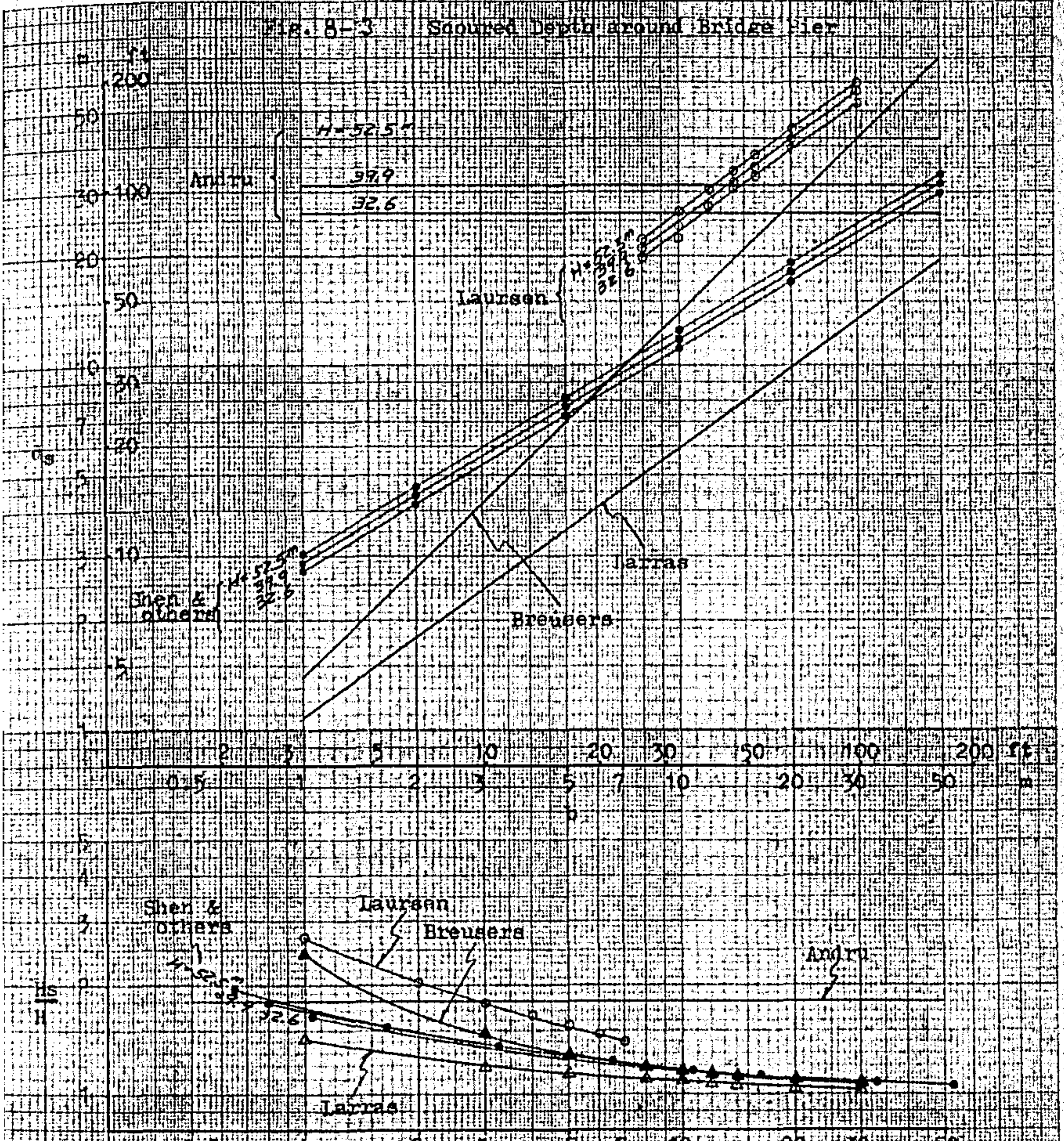


Fig. 8-3 Scoured Depth around Bridge Pier
Andru
H=52.5
39.9
32.6
Laursen
Shen & others
Larras
Breusers
ft
m
b
Hs/H
H/b
Andru : Hs/H = 1.6
Breusors: ds = 1.4b
Larras : ds = 1.05b^0.75 (m)
Shen & others : ds = 0.000223Re^0.619 (m)
bridge pier
b
H
Hs
ds

$k_\alpha = 1.0$, Laursen's formula becomes

$$H_s = H + 0.9\ k_\tau d_s \qquad (6)$$

since $u_*/w_o > 2$ in the case of the Jamuna River.

Values of $H_s/H$ calculated for $H/b = 1$ to 7 are shown in Fig. 8-4, which indicates that $H_s/H < 1.8$ for a range of $H/b > 3$ and $H_s/H > 1.8$ for a range of $H/b < 3$ according to Laursen's formula, while $H_s/H$ is always equal to 1.8 according to Andru's.

Depths of scour at the piers in each type of the constrictions at the proposed four sites were calculated by the two formulas mentioned above. The results are shown in Table 8-1. It is seen from this table that, in the types of A and B, the values calculated by Andru's formula are larger than by Laursen's and those by both formulas are almost equal for the type C. In consideration of these results, the values by Andru's are shown in Fig.7-8 by double-circle points.

3. Protection Works around Wider Piers.

Bridge piers should be put down deep enough to stand by themselves without any protection around them. If there are some reasons that this is very difficult or too uneconomical, we will be forced to consider some protection works although undesirable. Thinking of this condition, a study was made of range of protection and size of stones to be placed around piers. This may be called pier apron.

(1) Range for protection.

i. Gales' proposal

R.R.Gales proposed a protection range to be taken around piers in consideration of oblique attack of flow and scattering of stones in falling, which is shown in Fig. 8-5.

ii. K.Ishizaki and K.Honma's study.

Katsuyoshi Ishizaki and Katsuichi Honma (129 GB) obtained the results shown in Fig. 8-6 and 8-7 on the basis of extensive experiments. In these figures, H denotes normal water depth without scour, q and v unit discharge and mean velocity in the normal water depth, X distance from the pier wall, r radius of the pier, $d_s$ depth of scour at the pier wall or at X = 0, H' the scoured depth measured from the water surface, q' and v' unit discharge

Table 8 - 1 Estimation of Scoured Depth around Bridge Pier

$b_p = 12.0^m$

| Site | Type | Depth H (m) | Laursen's method | | | Andru's formula | | Design Water depth at piers |
|---|---|---|---|---|---|---|---|---|
| | | | H/$b_p$ | Hs/H | Hs (m) | Hs/H | Hs(m) | m (ft) |
| Nagarbari | A | 56.139 | 4.678 | 1.56 | 87.577 | 1.80 | 101.050 | 102(334.4) |
| | B | 42.384 | 3.532 | 1.69 | 71.629 | " | 76.291 | 77(252.5) |
| | C | 36.825 | 3.069 | 1.77 | 65.180 | " | 66.285 | 67(219.7) |
| Sirajganj | A | 53.862 | 4.489 | 1.57 | 84.563 | " | 96.952 | 97(318.0) |
| | B | 40.749 | 3.396 | 1.71 | 69.681 | " | 73.348 | 74(242.6) |
| | C | 33.473 | 2.789 | 1.82 | 60.921 | " | 60.251 | 61(200.0) |
| Gabargaon | A | 54.891 | 4.574 | 1.56 | 85.630 | " | 98.804 | 99(324.6) |
| | B | 41.041 | 3.420 | 1.70 | 69.770 | " | 73.874 | 74(242.6) |
| | C | 35.554 | 2.963 | 1.80 | 63.997 | " | 63.997 | 64(209.8) |
| Bahadurabad | A | 50.542 | 4.212 | 1.60 | 80.867 | " | 90.976 | 91(298.4) |
| | B | 37.682 | 3.140 | 1.75 | 65.944 | " | 67.828 | 68(223.0) |
| | C | 31.229 | 2.602 | 1.86 | 58.086 | " | 56.212 | 57(186.9) |

Fig. 8-5

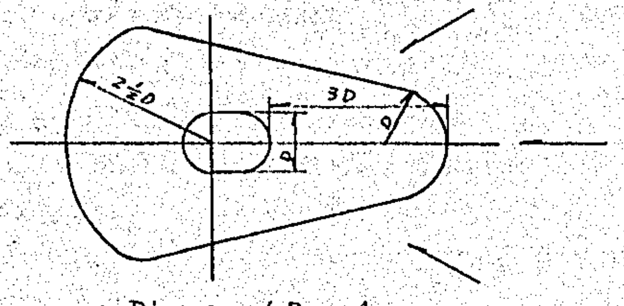



Diagram of Pier-Apron

Fig. 8-6

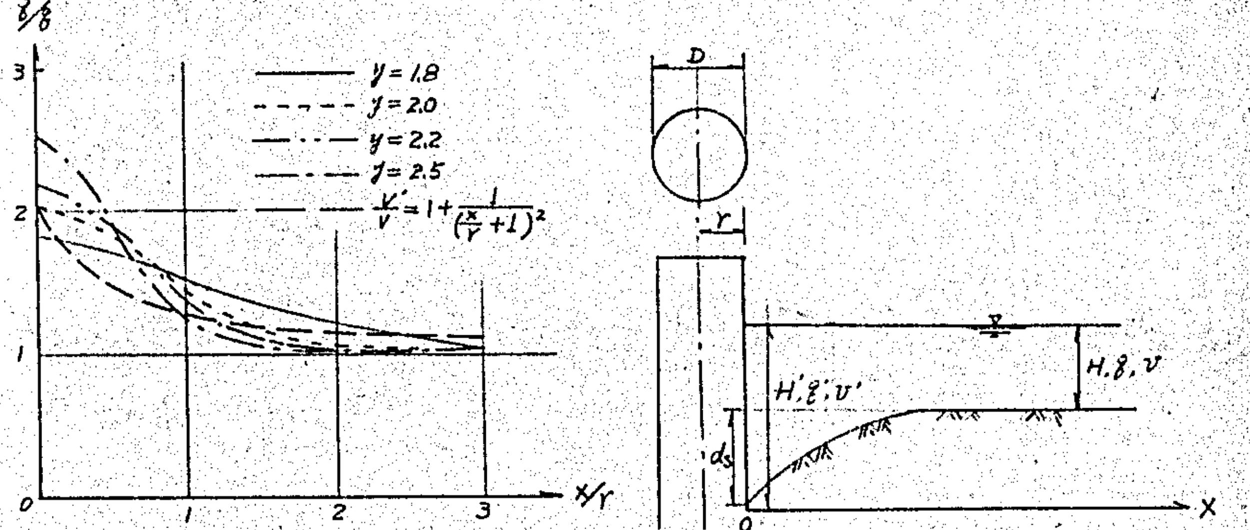
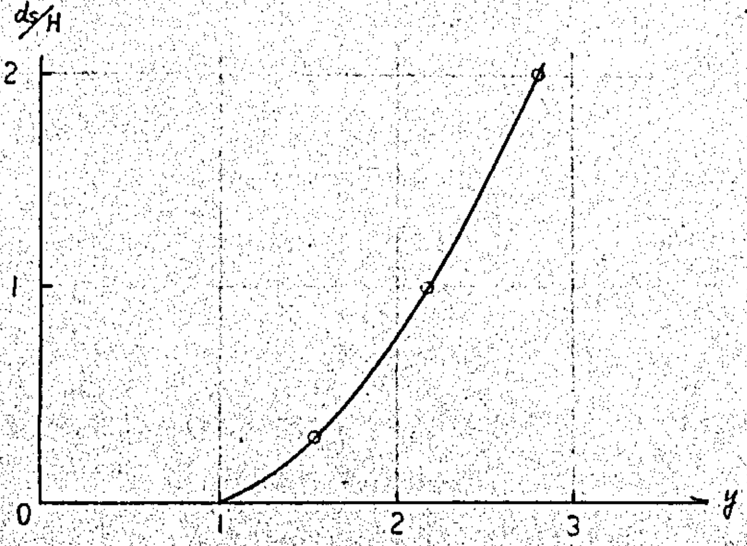



Fig. 8-7



and mean velocity at the pier wall, and y a value of q'/q at the pier wall or at X = 0. Fig. 8-6 shows the correlation between q'/q and X/r, and Fig. 8-7 shows the correlation between $d_s/H$ and y.

As seen in Fig. 8-6, the ratio q'/q is larger than unity in the range X/r<2 or in the distance smaller than D from the pier wall. Therefore, it may be sufficient, if we take the range for protection as 2D from the pier wall in consideration of scattering of stones, where D is diameter of pier.

iii. Present state at Hardinge Bridge.

According to the results of pricking around the piers of Hardinge Bridge, it is reported that stones have been scattered around the piers approximately within the range of about 2D from the pier wall.

(2) Size of stones.

S.V.Isbash studied the stability of rock-fill dam in running water and obtained the following formula describing the relation between flow velocity and weight of stones required for stability.

$$v = EK\sqrt{d} \qquad (7)$$

$$K = \sqrt{2g(w_s - w_o)/w_o}$$

where $v$ = critical mean velocity (mean velocity on a vertical) (m/s)
$d$ = diameter of a stone (m)
$w_s$ = unit weight of stone (t/cub. m)
$w_o$ = unit weight of water (t/cub. m)
$g$ = acceleration of gravity (m/sq. s)
$E$ = coefficient to be used depending to shape of the crest of dam (nondimension).

The values of coefficient E are

E = 0.86 for triangular crest

E = 1.20 for trapezoidal crest.

In a local scour as shown in Fig. 8-6,

$$v = q/H, \quad v' = q'/H'$$

$$\therefore \quad v' = (H/H')yv \qquad (8)$$

Substituting the equation (8) into the equation (7), we get

$$d = (H/H')^2(y/EK)^2v^2. \qquad (9)$$

Therefore, we can estimate the size of a stone which will not be moved by the flow in the scour hole, if we can get the values of scoured depth, unit discharge at the pier wall and the mean velocity at the normal depth. Weight of the stone can be approximately estimated by the following equation.

$$W = 2.65(4/3)\pi(d/2)^3 \qquad (10)$$

Regarding the mean velocity v, we must take the velocity at the thalweg. Now, let v be the mean velocity at the thalweg and $v_M$ the mean velocity of the whole water area, we get

$$v = (1/n)H^{2/3}I^{1/2}, \quad v_M = (1/n_M)R^{2/3}I_M^{1/2}$$

where n is coefficient of roughness, H water depth and I water-surface slope at the thalweg and $n_M$ is coefficient of roughness, R mean depth and $I_M$ water surface slope of the whole cross-sectional area. If we can approximately put

$$n = n_M, \quad I = I_M$$

the mean velocity at the thalweg between the both guide banks can be estimated by the following equation.

$$v = (H/R)^{2/3}v_M \qquad (11)$$

Now, if we take the values, $g = 9.8\ m/s^2$, $w_s = 2.65\ t/m^3$, $w_o = 1\ t/m^3$, and E = 1.20, the equation (9) becomes

$$d = 0.0215(H/H')^2y^2v^2. \qquad (12)$$

According to the study made by Ishizaki and Honma, the relation between $d_s/H$ and y is given by Fig. 8-7. If we take $H'/H = 1.8$ or $d_s/H = 0.8$, we get y = 2.05 from Fig. 8-7. On the other hand, the theoretical solution gives

$$v'/v = 1 + (X/r + 1)^{-2}$$

for the state of non-scour, or y = 2 at the pier wall. Therefore, we can understand that the value of y varies from 2 to 2.05 depending on the state of scour, from the beginning to the equilibrium, on the assumption that $H'/H = 1.8$ or $d_s/H = 0.8$.

Since the value of y at the equilibrium state of scour scarecely differs from that at the beginning of scour, we can use the value y = 2.05 for the intermediate state of scour. In this case, the equation (12) becomes

$$d = 0.09035(H/H')^2v^2. \qquad (13)$$

Tables 8-2-1 to 8-2-4 give the values of diameter d and weight W of the stones which will resist to the flow expected to occur at the bottom of prospective scour hole. For this calculation, the equations (13) and (10) were used and, as mentioned above, H means water depth at a thalweg, $H_s$ the water depth from the water surface to the bottom of the scour in the equilibrium state and H' a water depth of the scour in a state on the way to the equilibrium.

It will be considered on the basis of the results of the calculation shown in Table 8-2-1 to 8-2-4 as follows.

a. Theoretically, it is necessary to place stones larger than those shown below so as not to allow any scour around piers. In practice, stones smaller than these may serve if necessary supply for maintenance is considered.

| | | | |
|---|---|---|---|
| Type A: | Nagarbari site | d = 135 cm | w = 3,430 kg |
| | Sirajganj site | d = 171 cm | w = 6,959 kg |
| | Gabargaon site | d = 166 cm | w = 6,370 kg |
| | Bahadurabad site | d = 196 cm | w = 10,479 kg |
| Type B: | Nagarbari site | d = 115 cm | w = 2,120 kg |
| | Sirajganj site | d = 146 cm | w = 4,343 kg |
| | Gabargaon site | d = 144 cm | w = 4,160 kg |
| | Bahadurabad site | d = 171 cm | w = 6,960 kg |
| Type C: | Nagarbari site | d = 100 cm | w = 1,380 kg |
| | Sirajganj site | d = 122 cm | w = 2,518 kg |
| | Gabargaon site | d = 125 cm | w = 2,740 kg |
| | Bahadurabad site | d = 140 cm | w = 3,820 kg |

b. If a pier be put down deep enough to have a required grip length from the bottom of the equilibrium scour hole, it is of course unnecessary to place stones in the hole, because, in the state of equilibrium, the scoured hole will be supplied with sediment by water flow and hold a definite equilibrium depth. The values given in the tables mean the sizes of stones which will not be moved without the supply of sediment.

c. If we want to reduce scoured depth or hold a depth H' smaller than $H_s$, it will be necessary to place stones of the order of sizes calculated in the tables with regard to scoured depth H'. Also in this case, stones smaller than these will serve if we consider the continuous supply of sediment.

Table 8-2-1 Weight of Stones

Site: Nagarbari

| Type | B (m) | $v_m$ (m/s) | H (m) | $v$ (m/s) | H' (m) | H/H' | d (m) | W (t) |
|---|---|---|---|---|---|---|---|---|
| | | | | | Hs = 101.050 | Hs/H = 1.8 | | |
| A | 2,000 | 1.993 | 56.139 | 3.871 | 56.139 | 1 | 1.352 | 3.429 |
| | | | | | 67 | 0.838 | 0.950 | 1.190 |
| | | | | | 78 | 0.720 | 0.701 | 0.480 |
| | | | | | 89 | 0.631 | 0.538 | 0.216 |
| | | | | | 101,050 | 0.556 | 0.418 | 0.101 |
| | | | | | Hs = 76.291 | Hs/H = 1.8 | | |
| B | 4,200 | 1.580 | 42.384 | 3.573 | 42,384 | 1 | 1.152 | 2.121 |
| | | | | | 50 | 0.848 | 0.828 | 0.788 |
| | | | | | 59 | 0.718 | 0.594 | 0.291 |
| | | | | | 68 | 0.623 | 0.447 | 0.125 |
| | | | | | 76.291 | 0.556 | 0.356 | 0.063 |
| | | | | | Hs = 66.285 | Hs/H = 1.8 | | |
| C | 5,200 | 1.468 | 36.825 | 3.319 | 36,825 | 1 | 0.997 | 1.379 |
| | | | | | 44 | 0.837 | 0.696 | 0.468 |
| | | | | | 51 | 0.722 | 0.518 | 0.193 |
| | | | | | 58 | 0.635 | 0.401 | 0.090 |
| | | | | | 66,285 | 0.556 | 0.307 | 0.041 |

Table 8-2-2 Weight of Stones

Site: Sirajganj

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | Hs = 96.952 | Hs/H = 1.8 | | |
| A | 2,000 | 2.246 | 53.862 | 4.356 | 53,862 | 1 | 1.712 | 6.959 |
| | | | | | 60 | 0.898 | 1.381 | 3.653 |
| | | | | | 70 | 0.769 | 1.013 | 1.442 |
| | | | | | 80 | 0.673 | 0.776 | 0.648 |
| | | | | | 96,952 | 0.556 | 0.529 | 0.205 |
| | | | | | Hs = 73.348 | Hs/H = 1.8 | | |
| B | 4,200 | 1.780 | 40.749 | 4.026 | 40,749 | 1 | 1.463 | 4.343 |
| | | | | | 50 | 0.815 | 0.971 | 1.270 |
| | | | | | 60 | 0.679 | 0.674 | 0.425 |
| | | | | | 70 | 0.582 | 0.495 | 0.168 |
| | | | | | 73,348 | 0.556 | 0.452 | 0.128 |
| | | | | | Hs = 60.251 | Hs/H = 1.8 | | |
| C | 5,600 | 1.625 | 33.473 | 3.676 | 33,473 | 1 | 1.220 | 2.518 |
| | | | | | 40 | 0.837 | 0.854 | 0.864 |
| | | | | | 50 | 0.669 | 0.546 | 0.226 |
| | | | | | 60 | 0.558 | 0.380 | 0.076 |
| | | | | | 60,251 | 0.556 | 0.377 | 0.074 |

Table 8-2-3 Weight of stones

Site: Gabargaon

| Type | B (m) | $v_m$ (m/s) | H (m) | $v$ (m/s) | H' (m) | H/H' | d (m) | W (m) |
|---|---|---|---|---|---|---|---|---|
| A | 2,000 | 2.209 | 54.891 | 4.291 | Hs = 98.804 Hs/H = 1.8 | | | |
| | | | | | 54.891 | 1 | 1.662 | 6.370 |
| | | | | | 65 | 0.844 | 1.184 | 2.303 |
| | | | | | 76 | 0.722 | 0.866 | 0.901 |
| | | | | | 87 | 0.631 | 0.662 | 0.403 |
| | | | | | 98.804 | 0.556 | 0.514 | 0.188 |
| B | 4,200 | 1.767 | 41.041 | 3.997 | Hs = 73.874 Hs/H = 1.8 | | | |
| | | | | | 41.041 | 1 | 1.442 | 4.160 |
| | | | | | 49 | 0.838 | 1.012 | 1.438 |
| | | | | | 57 | 0.720 | 0.748 | 0.581 |
| | | | | | 65 | 0.631 | 0.574 | 0.262 |
| | | | | | 73.874 | 0.556 | 0.446 | 0.123 |
| C | 5,200 | 1.648 | 35.554 | 3.728 | Hs = 63.997 Hs/H = 1.8 | | | |
| | | | | | 35.554 | 1 | 1.254 | 2.736 |
| | | | | | 42 | 0.847 | 0.900 | 1.011 |
| | | | | | 49 | 0.726 | 0.661 | 0.403 |
| | | | | | 56 | 0.635 | 0.506 | 0.180 |
| | | | | | 63.997 | 0.556 | 0.388 | 0.081 |

Table 8-2-4 Weight of stones

Site: Bahadurabad

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| S | 2,000 | 2.400 | 50.542 | 4.662 | Hs = 90.976 Hs/H = 1.8 | | | |
| | | | | | 50.542 | 1 | 1.961 | 10.479 |
| | | | | | 60 | 0.842 | 1.391 | 3.807 |
| | | | | | 70 | 0.722 | 1.022 | 1.481 |
| | | | | | 80 | 0.632 | 0.783 | 0.669 |
| | | | | | 90.976 | 0.556 | 0.606 | 0.309 |
| B | 4,200 | 1.925 | 37.682 | 4.354 | Hs = 67.828 Hs/H = 1.8 | | | |
| | | | | | 36.682 | 1 | 1.711 | 6.962 |
| | | | | | 45 | 0.837 | 1.199 | 2.400 |
| | | | | | 52 | 0.725 | 0.899 | 1.011 |
| | | | | | 60 | 0.628 | 0.675 | 0.429 |
| | | | | | 67.828 | 0.556 | 0.529 | 0.207 |
| C | 5,600 | 1.742 | 31.229 | 3.940 | Hs = 56.212 Hs/H = 1.8 | | | |
| | | | | | 31.229 | 1 | 1.401 | 3.824 |
| | | | | | 37 | 0.844 | 0.998 | 1.379 |
| | | | | | 43 | 0.726 | 0.738 | 0.558 |
| | | | | | 50 | 0.625 | 0.547 | 0.228 |
| | | | | | 56.212 | 0.556 | 0.433 | 0.113 |

to the hole and maintenance supplement of stones.

d. Since the sizes of stones given in the tables are those capable of resisting to the maximum velocity at the pier wall, it may be allowed to use smaller stones on the downstream side of the pier.

(3) Thickness of stones to be placed.

In our judgement, it will be necessary to provide the same thickness as in the case of the apron in the meaning of preventing the leakage of sand through voids of the placed stones.