3-3 River Improvement Plan

1. Basic Policy

The range of the river improvement range will be 16.4 km from the constrict portion of STA.16+480 to the confluent portion of the Gua-Gua River at the down stream. This plan will be established by the following basic policies.

(i) Existing river course will be highly considered.

(ii) A pilot channel will be established at the center of the river-course, and the probability for this flow will be about 1 year.

(iii) For the longitudinal section, natural river-system will be used as in mild slope at the down stream and steep slope at the upper stream.

(iv) For the bank-height, the cross-sectional area of the river will be secured for the equation of Q (design discharge) = 900 m^3/s with the probability of 80 years, and the free-board will be 1.50 m in view of the safety for settlement to consolidation.

2. Alignment

With the diversion of the existing bank of BPW-plan, the bank-standard line will be extended by BPW-plan for the section from non-accomplished part STA.2+350 to the down stream of Gua-Gua River in a distance of 2.35 km and the section from STA.13 + 50 of upper stream from Mitala to STA.15+950 in a distance of 2.90 km (from STA.14+40 at the left bank in a distance of 1.60 km). However, as to the left embankment, the site around STA.16 is constrict, and both sides of the banks are rather strong with hard rock, thus the width of the river-course will be extended up to this site.

Moreover, a branch river flows at both sides of the banks at the upper stream of Mitla, and the braining of landside water with natural control will be done under the plan of the open levee of the bank because of the slope of the river bed over 1/100.

(1) How to consider the bank alignment

1) Flood flows in a streight line, thus the alignment of the bank will be established along the stream-line as much as possible, and the radial at bent part will be over 5 times that of river-width.

2) River-width will be considered with the planned flooding flow, and high water channel will be taken as big as possible, however, too big (wide) width will cause deflecting or turbulant streams.

Therefore, the following formula will be considered.

$$B = 3.5 - 7.0Q^{1/2}$$

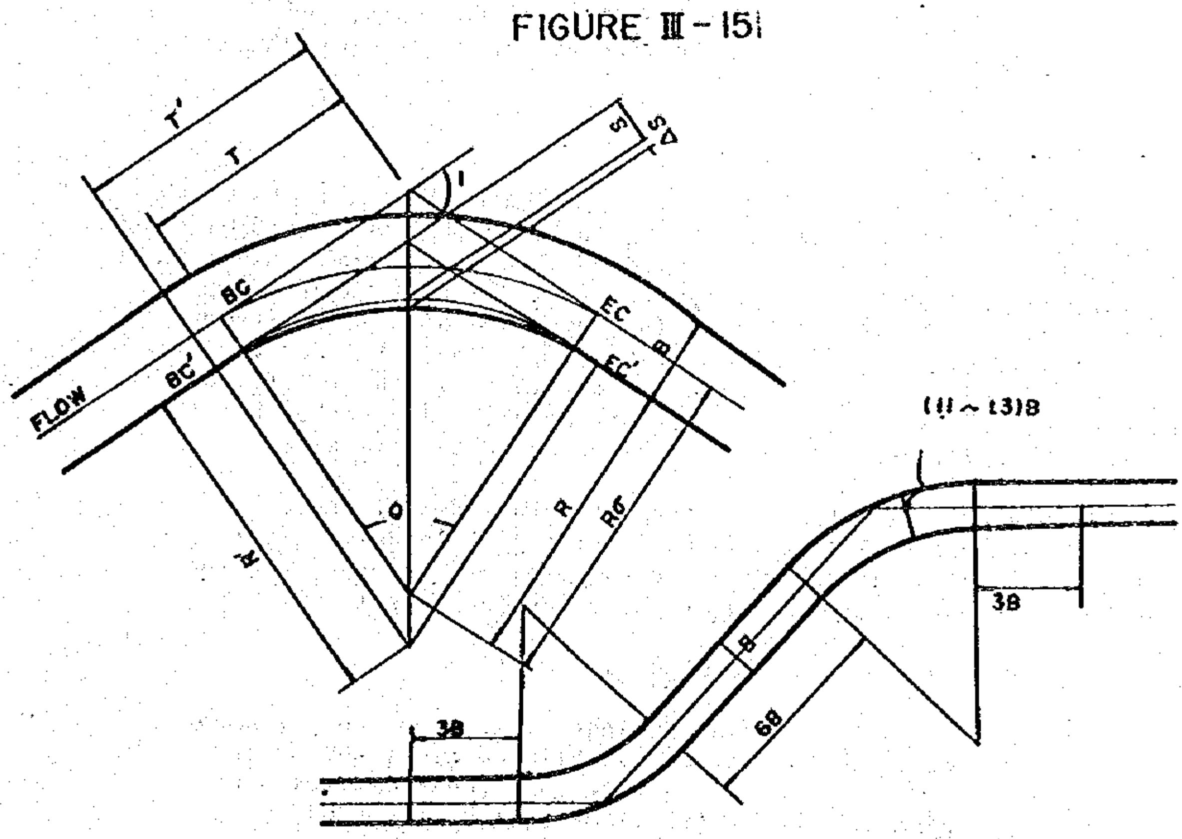
Also, at the smaller river-width, side erosion is liable to occur, and flow-route is liable to be extended.

3) A sudden change of river-width is liable to cause disasters, and the confluence will be desired to take acute angle as much as possible.

4) According to the air-photo of Pasig-Potrero River, a certain interval is observed in the river course, that is, hitting is observed to the bank with a pitch about 500 m.

5) Width-extension at the bent portion will be done by the following FIGURE IV-15.

FIGURE III - 15



6) At the open levee, the bank is disconnected, thus the water enters into the bank from the open part with counter-current, but the flood-continuing time is short, and the ground in the bank is declined with a higher slope towards the upper stream, thus the counter-flow is less and the damage will be a little.

Also, some flood-controlling effects will be observed, hile the flow-route is liable to be getting along with the projecting part of the open levee, therefore, it is necessary to maintain the flow-route and enforcement of the dike.

7) As for the drainage sluice pipe crossing the bank, the locality will be at the place of water-way or at the lowest ground-site without water-way, and installation will be made in crossing the bank-normal line rec-rectangularly.

(2) Lower water-way region

Lower-water ways are liable to show deflecting or turbulent currents, and liable to be meandering, thus the flow-target of 120 m^3/s will be planned with the probability of one year. The locality for construction will be extended to the whole line of the river course STA.15+950m with the center of the existing bank-inside.

The river-width will be $B = 3.5\ Q^{\frac{1}{2}} \doteqdot 38$ m .

Accordingly, the channel-width will be 30 m in its trapezoid section and the present river-bed will be excavated.

There are bridges at the Highway By-Pass and the national road part of Bacolor-down stream, and the slope of the river-bed is mild, and the limiting excavation-depth of the pier is restricted to be 2m, thus the channel width for the section of 4.3 km from STA.4 + 300 to the estuary of the down stream will be 60m-trapezoid section.

There is a wider high-water channel at the upper stream sround STA.7, thus a section of maintenance-river bed at the lateral direction shall be established with the minimum 50 m from the end of the bank-normal line. Width-extension to the lateral direction is impossible at the down stream from STA.2+150m, and some margins will be taken under the lower water-channels with the excavation of 0.5 - 1.0 m.

3. Longitudinal profile and cross-section

The present river slope is mild at the down stream. It is a natural state with the steep slope at the upper stream.

The existing river course will be used in diversion, and lower water-ways will be excavated at the central part with the depth within 2.5 m, and the excavated limiting depth will be 2.0 m at the pier of the bridges at Highway By-Pass and a national road at Bacolor.

The margin of 1.0 m is enough for the crown-height of the bank at the planning flood flow. Whereas, a marginal height of 1.5 m will be considered in view of the safety of the existing bank and the compacting subsidence of the bank-body.

(1) General way of consideration

i) The river slope is milder at the down stream than at upper stream, and wave-form movement occurs with the interchanging occurrence of washing excavation and sedimentation when the flooding wave length is large.

ii) In restricting the plane shape to (at) the down stream, a transitional part of the slope of river-bed occurs at the upper stream with the observation of abnormally sedimenting trend.

iii) The river-bed at flooding peak period will become soaked-excavating state at water-increasing period and back filling sedimentation at the water-decreasing period.

iv) The river-sectional view will be desirable to make double sections. However, it is difficult to maintain the lower water ways at the river of steep slope, thus there are many cases of single section.

(2) Longitudinal profile and cross-section

The existing river course from the Gua-Gua River to STA.16 point has the section being above to flow the planned flooding flow Q = 900 m^3/s. As for the excavating river course at lower water ways, the following items will be adopted.

i) With the trapezoid section excavating the present river course, the side slope will be 3 : 1, and the maximum excavating depth will be within 3 m.

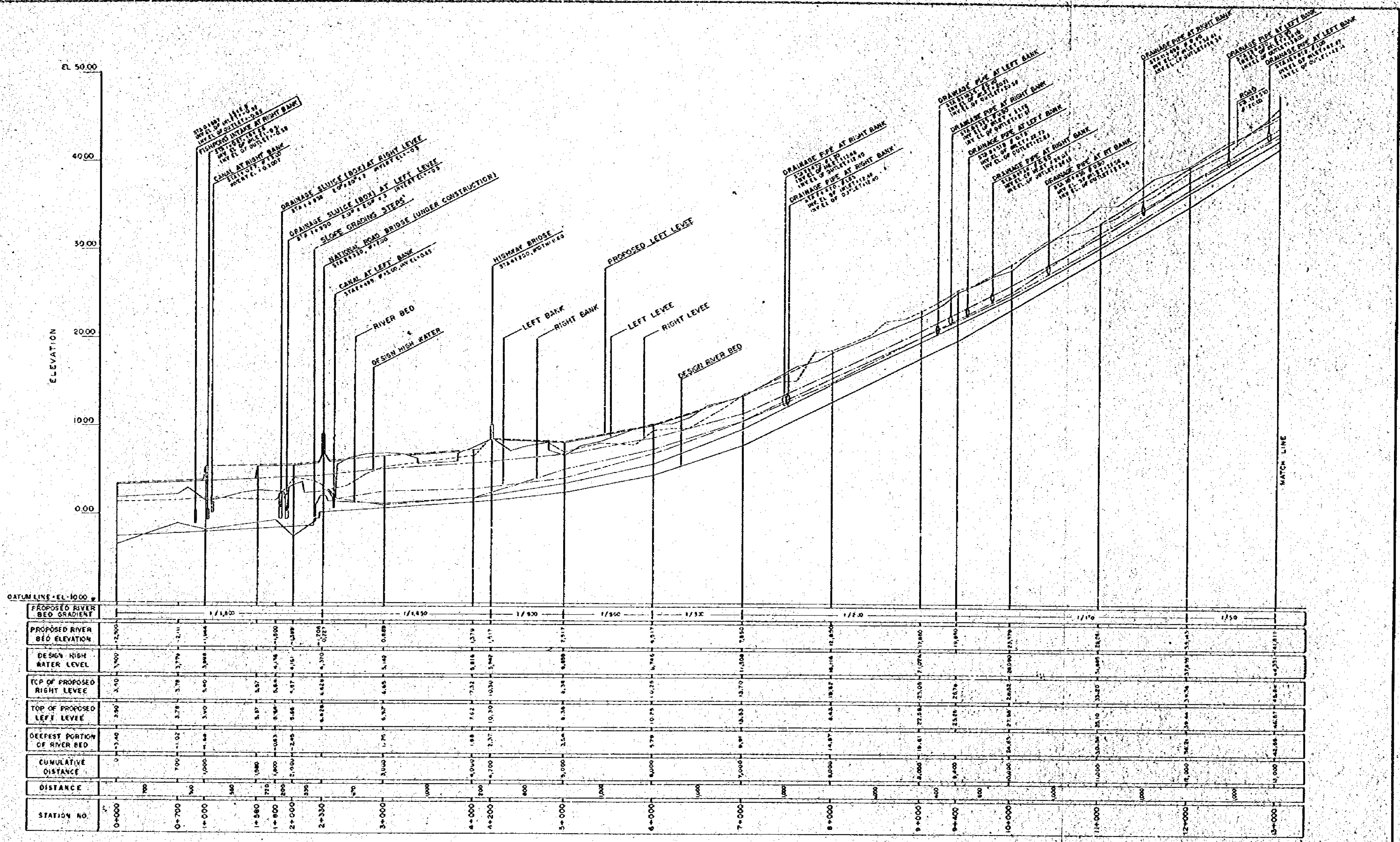
ii) The section shall be that flowing the object-flow Q = 120 m^3/s with the probability of 1.1 year.

iii) The channel-width at lower water-ways will be 60 m from STA.4+300 to the confluence of the Gua-Gua River, and will be 30 m at the upper stream. This transition-section will be 200 m.

iv) Excavating limit of the depth will be 2.0 m at Highway By-pass and at the pier of Bacolor. Also, the pier will be surrounded by the marketed pipes for preventing the washing excavation.

v) For the embankment at the lower water ways, the alignment face will be protected for 15 m at the upper stream and 35 m at the lower stream by means of the Gabion cylinder.

vi) River-bed slope at the down stream of National Road at Bacolor will form the transitional part to the milder slope, whereas the excavating will be limited for the



SCALE HORIZONTAL 1 : 20000
VERTICAL 1 : 200

FIGURE III-16-1 PROFILES

DPWTC
TASK FORCE FOR FLOOD CONTROL AND RELATED ACTIVITIES
PASIG-POTRERO RIVER FLOOD CONTROL AND SABO PROJECT PHILIPPINES
SEP 30,1978 AS SHOWN
PPFS 108
PROFILE 2 - 1
JAPAN INTERNATIONAL COOPERATION AGENCY

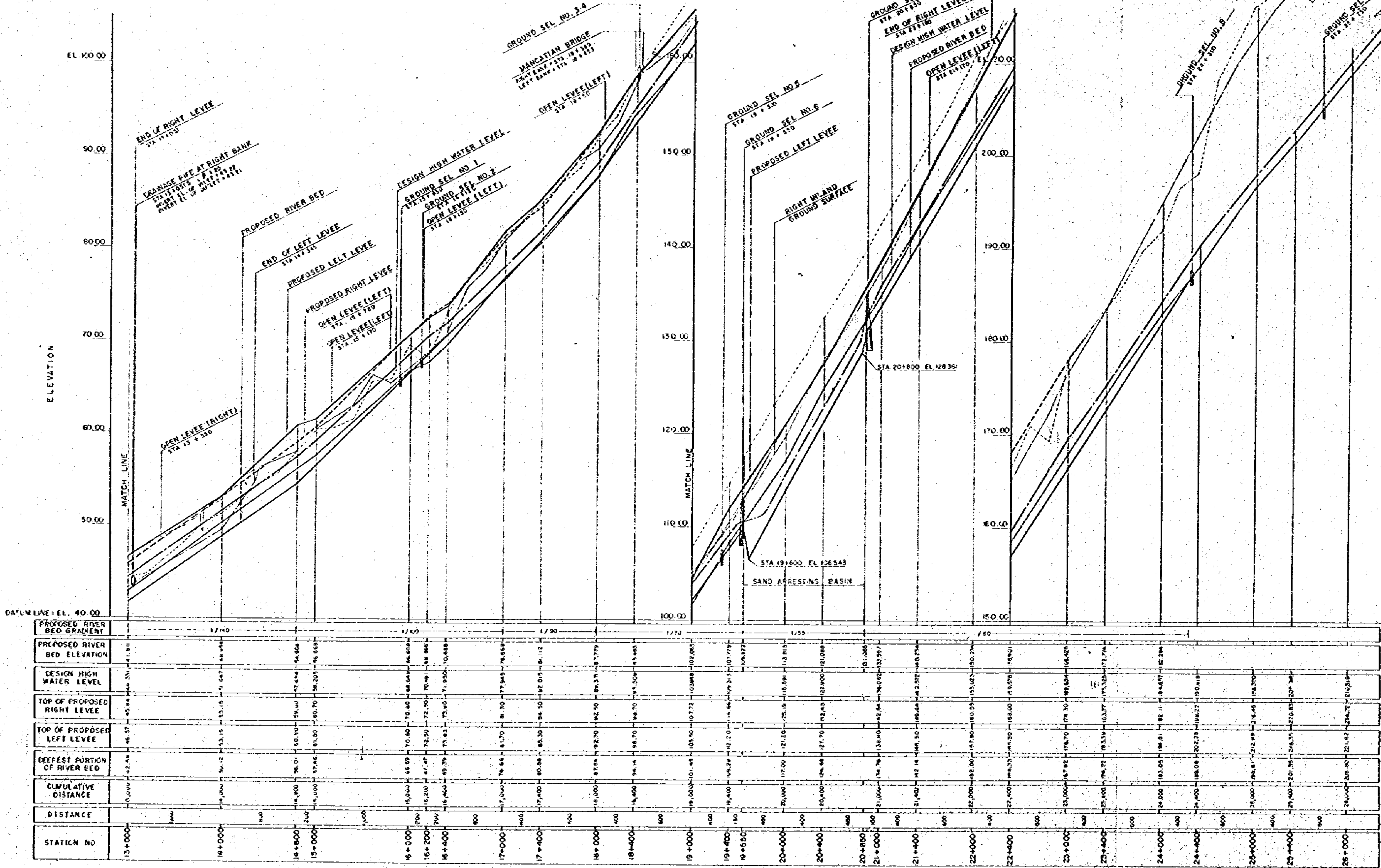


FIGURE Ⅲ-16-2 PROFILES

SCALE HORIZONTAL 1 : 20000
VERTICAL 1 : 200

DPWTC
TASK FORCE FOR FLOOD CONTROL AND RELATED ACTIVITIES

PASIG-POTRERO RIVER FLOOD CONTROL AND SABO PROJECT PHILIPPINES

SEP. 30, 1978

PROFILE 2 - 2

PPFS 109

JAPAN INTERNATIONAL COOPERATION AGENCY

pier, thus 2 stages' fall-work will be made within 150 m at the down stream of the pier.

(3) Determination of section at lower water channel

For the section of lower water way, trapezoid section will be taken for 2 cases of channel-width B = 30 m and 60 m. The section is determined with Manning Formula.

$$Q = A.v$$
$$= A.\frac{1}{n}.R^{2/3}.I^{1/2}$$

where,

Q: Discharge = 120 m^3/s

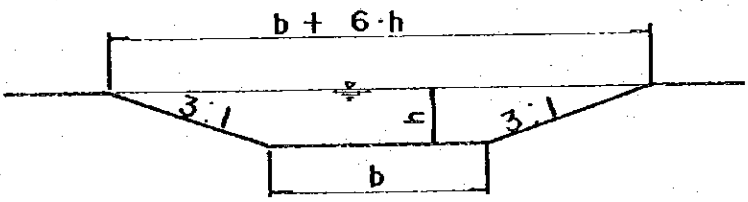
A: Discharge Area = (B + 3h).h

n: Coefficient of Roughness = 0.033

R: Hydroulic Radius = A/S (S: Penetrate length)

I: Gradient of river bed

FIGURE III-17



For the above 2 cases, B = 30 and 60 m, water-depth h and slope of river-bed I were employed as parameters.

The results of calculation are given in the following table.

FIGURE III-18-1
UNIFORM FLOW CALCULATION OF TRAPEZOID SECTION

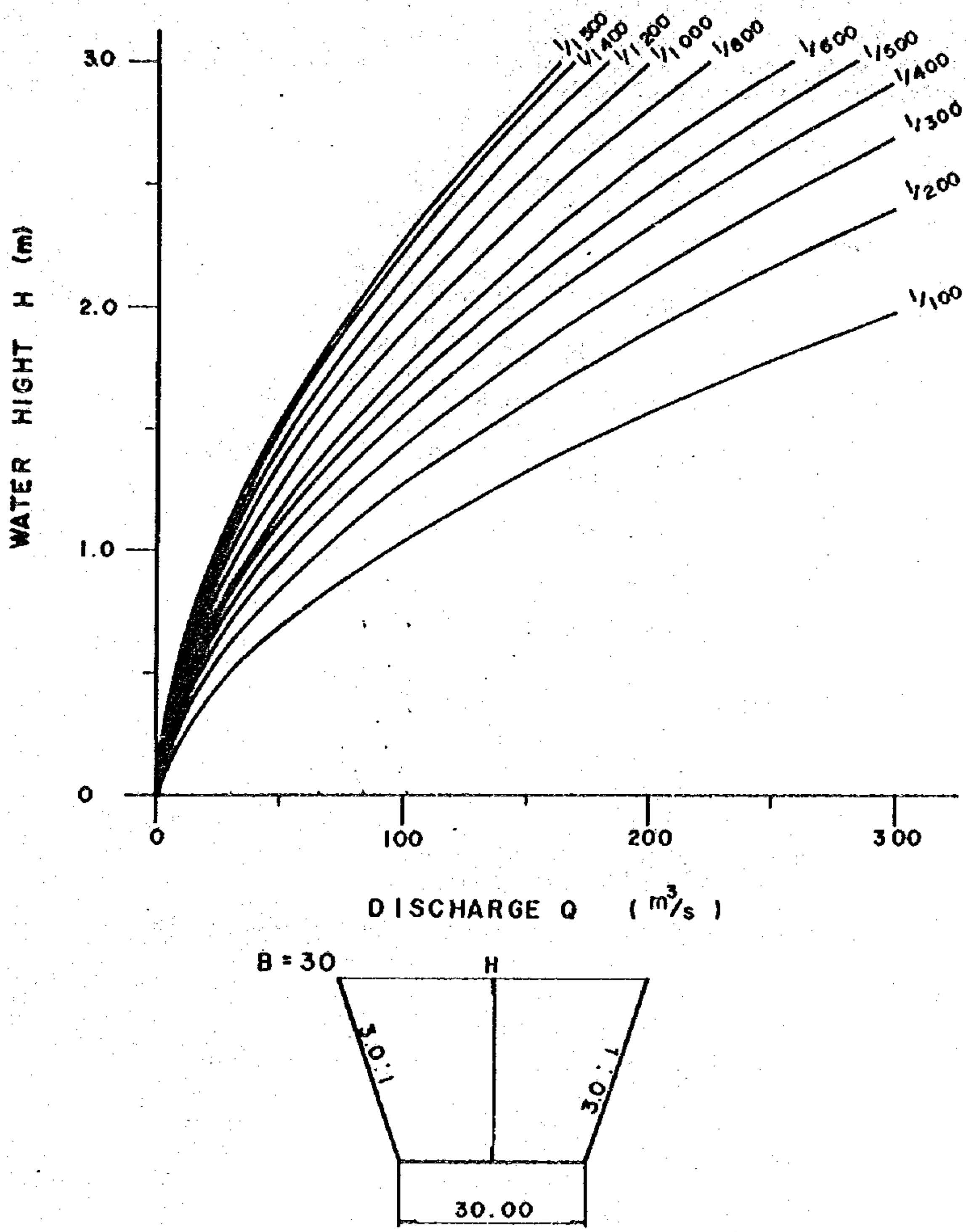
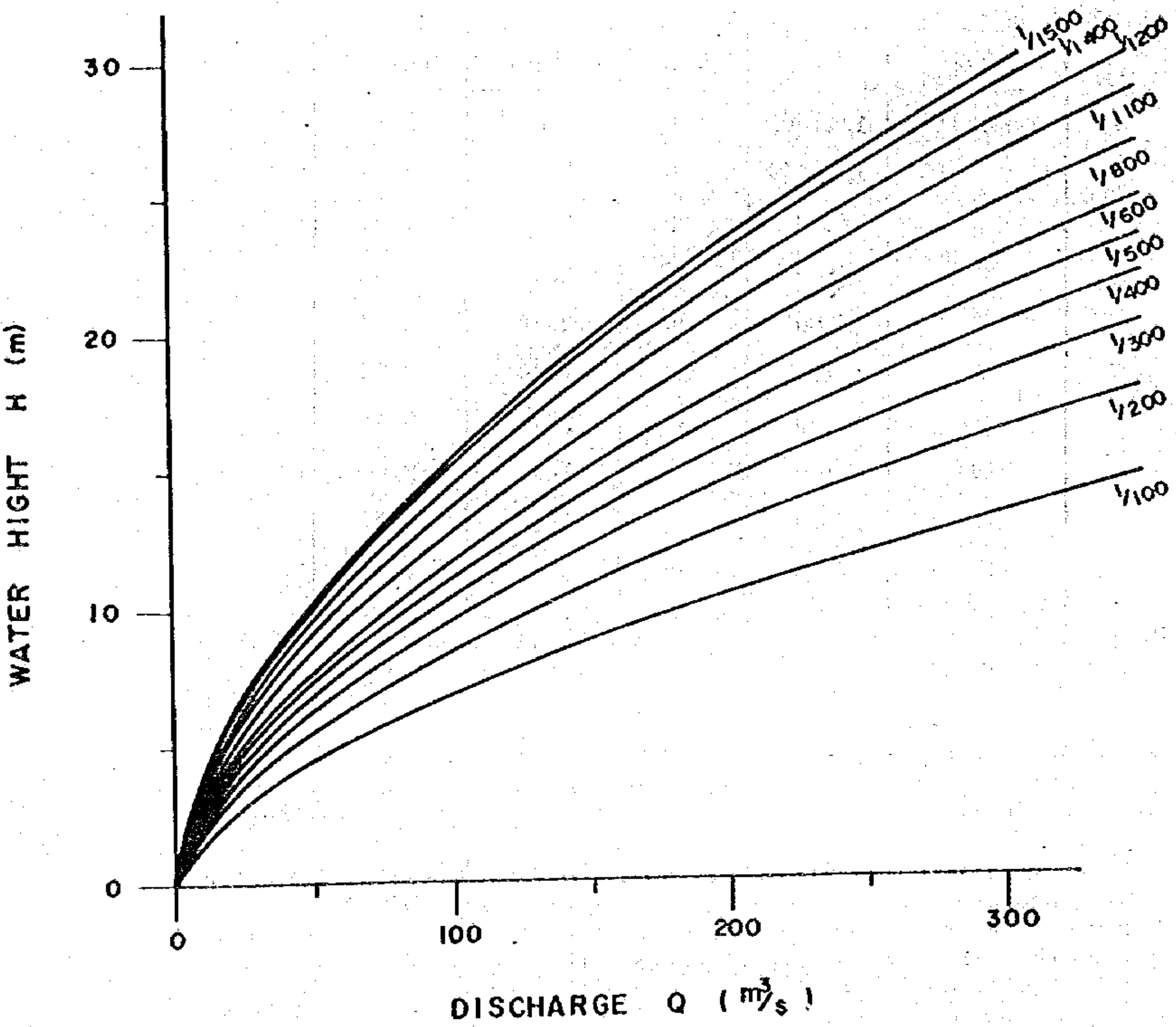


FIGURE Ⅲ-18-2.
UNIFORM FLOW CALCULATION OF TRAPEZOID SECTION



B = 60

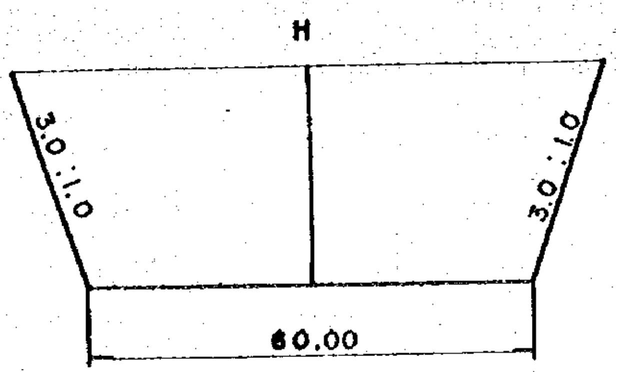


TABLE III-3 ; UNIFORM FLOW

STA	I	b (m)	h (m)	V (m/S)
2 + 330 m	1/1,800	60	2.3	1.16
4 + 220 m	1/1,450	60	1.3	0.91
5	1/800	30	1.0	1.01
6	1/500	30	1.5	1.62
7	1/300	30	2.10	2.55
8	1/200	30	2.20	3.21
9	1/200	30	2.1	3.12
10	1/170	30	1.3	2.55
11	1/170	30	1.3	2.55
12	1/170	30	1.5	2.78
13	1/150	30	1.1	2.46

4. Plan for the facilities

1. Levee

As mentioned in the above, (refer to 2.4.1), the bank at the Pasig-Potorero River has been gradually extended with the plan of B.P.W. At present, however, no completion is made for the whole river, thus the presnet reparian plan, the non-completed part of the bank will be extended.

In the present non-completed section, there is a part of no-bank and also there is a part without enough height of the bank although the embankment has been made.

Thus new embankment will be made for the part of no-bank, while banking side enforcement will be made for the part where the bank-height is insufficient, and these parts and sections are as follows.

TABLE III-4 EMBANKMENT

Division	Bank side	Section	Distance
New Embankment	Left	STA. 0+ 50 - STA. 2+500	2,450 m
		STA.14+500 - STA.16+ 0	1,500 m
	Right	STA. 0+ 50 - STA. 2+500	2,450 m
		STA.13+ 50 - STA.16+ 0	2,950 m
Levee Widening	Left	STA. 2+500 - STA. 6+550	4,050 m
		STA.14+500 - STA.16+ 0	1,500 m
	Right	STA. 2+500 - STA. 7+700	5,200 m
		STA.13+ 50 - STA.16+ 0	2,950 m

In order to avoid the decrease of the sectional area of river course, levee widening will be made within the banks.

The crown height of the bank shall be that added with a margin of 1.5 m to the water-level for the planned flow Q = 900 m^3/s. However, at present, no progress has been made for the riparian improvement of the main river, Gua-Gua, thus it is not necessary to complete the embankment at the lowest stream of the Pasig-potorero River, then, the temporary embankment will be made without marginal height.

The temporary banking section will be that without influence of the flood from the river to the houses directly, and decision was made as follows.

Left bank: Sta. 0+50 - Sta. 1+580

Right bank: Sta. 0+50 - Sta. 1+10

Moreover, for the unnecessary section of embankment for flow-drainage, where the ground-height of the river-side is enough higher than H.W.L., embankment with the height 0.5 m and width 6.0 m will be made for the maintenance road and boundery of river-side and land-side.

(2) Open levee

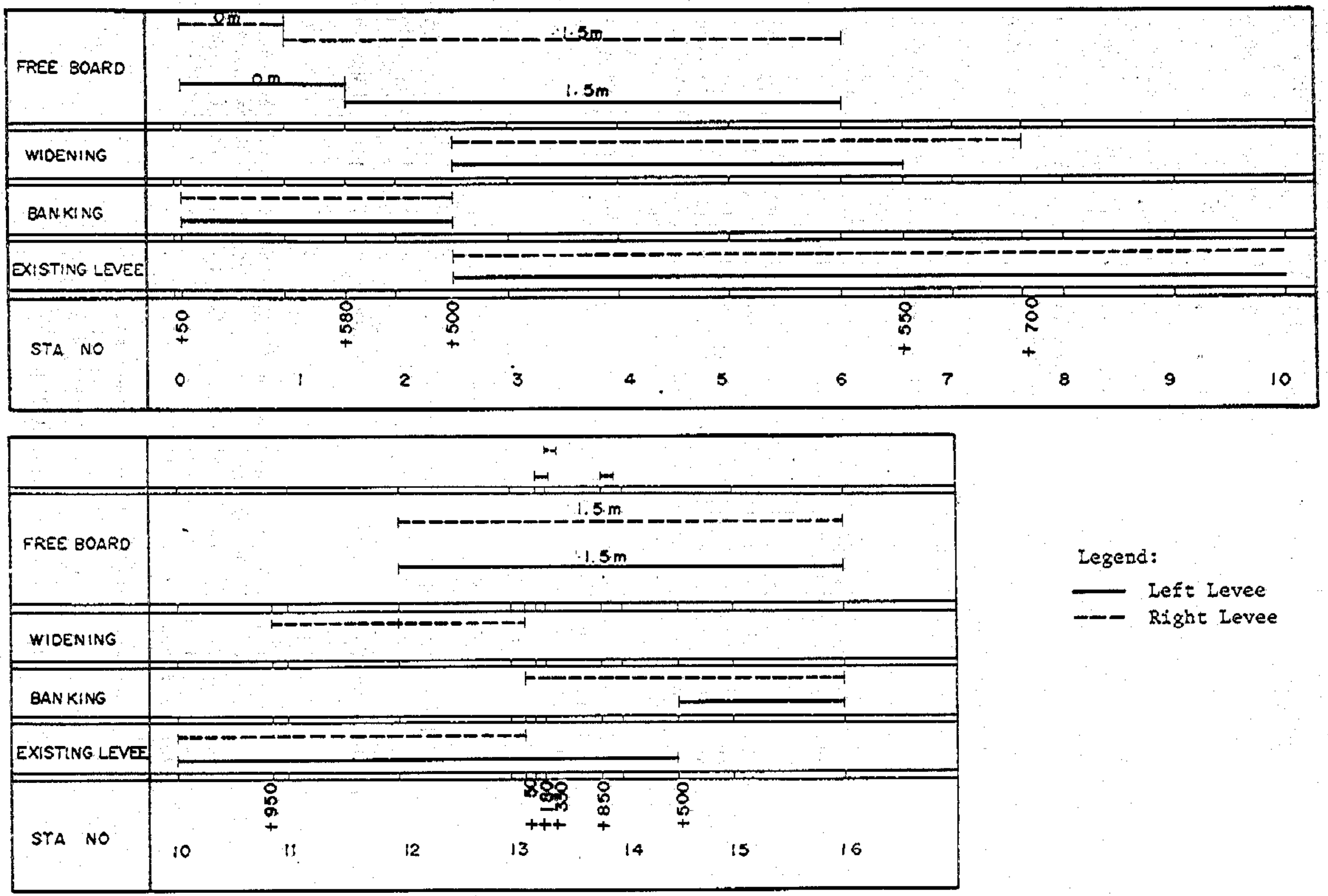
Open levee will be constructed at the left bank of down stream at the point of STA16 at the highest upper stream of river-course part and the right bank of STA13 + 300 for the draining of inside water as well as for securing the free-water functions against the abnormal flood exceeding the planned flow.

Arranged position of open levee is:

Left Bank: Sta.15+180, Sta.15+850

Right Bank: Sta.13+330

TABLE. III-5 LAYOUT OF EXISTING AND PROPOSED LEVEE



2. Revetment

1) Levee

In order to prevent the damage at flooding time, it is desirable to make banking revetment. Such existing works have been done with dry pitching for all the sections at the upper stream of Highway Br. (STA4+300), and this existing section will be utilized as it is.

Embarkment will be performed for the part where there is no revetment at STA2+500 - STA4+300 and the whole section for new construction of embankment.

2) Low water Channel

This revetment is desirable for the whole section as a rule, in order to fix the river course.

Whereas, in the major plan for riparian improvement, the following matters will be considered.

- o Sedimentation occurs at the river course by the supply of sediments from the upper stream in rainy season, thus it is necessary to remove the sediment at the river-bed at every dry season.

  Also, gravels are collected as materials for construction even from the river course, therefore, lower-water revetment will damage or disturb the collection and exclusion of sediments, and reversely, these works will induce unstable revetment.

- o Fixation of lower water way will be made by the maintenance and management of urgent restoring measures and for the exclusion of sediments, and it is desirable to reduce the construction-cost as much as possible.

With the above considerations, lower water way revetment will be arranged under the following policies.

- o The establishing section will be as little as possible, and it will be only the section for protection of the pier.

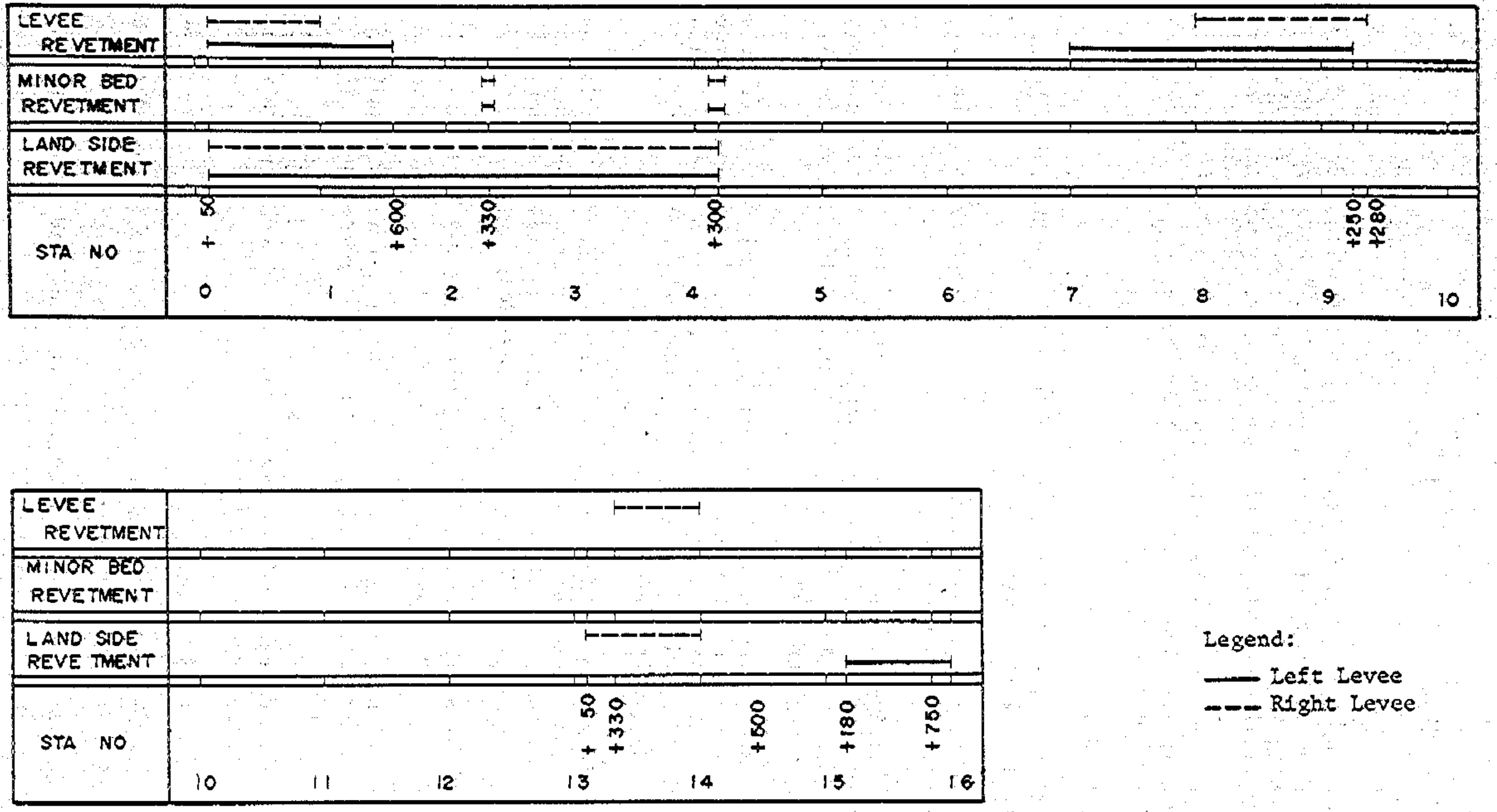
- o The concave side of river bent will be constructed with Goryne work.

3) Land-side

In the past result of damage of the bank, the damage of the bank was observed from the land-side slope.

Such sites will be protected with the land-side revetment for leg-part of the land-side inside slope of the bank by

TABLE III - 6



Legend:
—— Left Levee
- - - - Right Levee

installation of the back revetment.

Also, land-side revetment will be done for the prevention of the damage of the land-side slope due to the overflow as to the bank without freeboard at the river mouth.

TABLE III-7 LAND-SIDE REVETMENT

Bank Side	Section	Distance (km)
Left	STA. 0+ 50 - STA.1 +600	1.55
	STA. 7 - STA.9 +250	2.25
	STA. 0+ 50 - STA.1 + 0	0.95
	STA. 8 - STA.9 +280	1.28
	STA.13+330 - STA.14+ 0	0.67
Total		6.76

The above mentioned arrangement for the revetment of the river course is summarized in Table IV-4.

For the land-side slope, Grama grass or Gogon grass will be planted as vegetation.

3. Groyne

In order to protect the concave side of river bent of river course at the lower waterway, groyne will be established at the outside of the bent of waterway.

In view of statistic researches, the deep excavation at the bent will be made in the range from the end-point of the curve at the down stream to the distance of 2b in the right drawing with the starting point of the line of maximum depth hitting the river-side by the distance of b in the drawing, when river-width was regarded as b.

The groyne at the bent in the section of river course will be established in each section as follows, with the target of the range of deep excavating water.

FIGURE-III-19

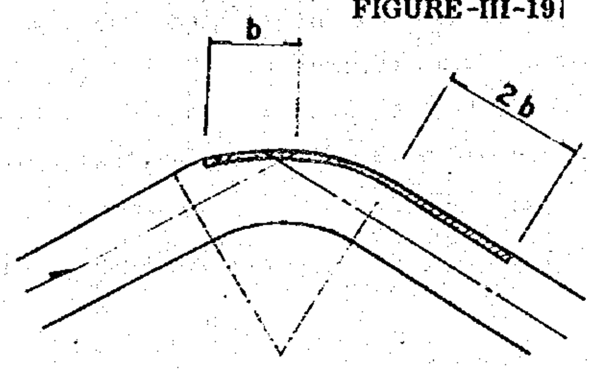


TABLE III-8 GROYNE

Bank Side	Section	Distance
Left	STA. 1+180 - STA. 1+830	650 m
	STA. 8+940 - STA. 9+400	460
	STA.14+170 - STA.14+710	540
	STA.15+680 - STA.15+840	160
Right	STA. 2+580 - STA. 3+ 50	470
	STA. 7+ 0 - STA. 7+680	680
	STA.12+240 - STA.12+790	550
	STA.15+680 - STA.15+840	160

4. River bed protection

The spot of Bacolor Br. (STA.2 330) is a dangerous site of washing excavation, because of shallow bridge-pier base, therefore, revetment with gabion cylinder will be done in a distance of 60 m at the upper stream and 110 m at down stream of the bridge.

The base of Bacorol Br. is a condition for deciding the longitudinal sectional shape of the river, that is, the excavating depth of river bed will be decided by the depth of the bridge-base.

Therefore, slope of river-bed at the down stream will be steeper than that of the river bed at the upper stream from the bridge. In order to avoid the unnaturalness of such slope of the river bed, the fall work at its height of 1.4 m will be established at the down stream from the bridge.

Concrete-structure will be unsuitable because the ground is soft, therefore, the fall-work will be done with 2 stages of 0.7 m within the range of the above mentioned revetment work.

3. 4 Sand Arresting Basin Project

1. Basic Principle

The sand arresting basin is to allow settlement of sediment discharge, beyond the control capacity of Sabo facilities such as Sabo dams in the up-stream area, over a part of the river channel in the down-stream area. The effects of the above is more prominently observed where an allowable sediment discharge in the down-stream are smaller.

In this Project, the basic principle of Sabo is to check up sediment at its original production area by constructing a group of Sabo dams over the up-stream mountain areas. However, due to topographical and geological conditions, there are limitations on location and maginitude of the sand arresting basins. It is practically impossible to regulate the entire sediment discharge by means of the group of Sabo dams. To compensate the sediment regulation volume of the group of dams and to prevent sediment discharge to the down-stream area, an excavated sand arresting basin is proposed under the principle as described below.

(1) Subject Section

The proposed site of the sand arresting basin is from the outlets of the mountain area to the extreme up-stream of the river channel, and the section on the plan is points between STA 26 + 498 (No. 1-A Groundsell Spot) and STA 15 + 950.

(2) Location of Sand Arresting Basin

The proposed sand arresting basin would be in the form of 1) widening of the river channel, and 2) excavation of the river bed, and after the completion, the required sand storage capacity may be secured by periodical excavation and the resulted earth moving.

Now, location of the proposed sand arresting basin would be:

+ where a sufficient required land is secured
+ where natural topographical conditions warrant a tendency to a smooth sedimentation
+ where hauling of earth from excavation is convenient

After taking the above into consideration, the location of the proposed sand arresting basin is determined at the fan-shaped delta in the up-stream of the Mancatian Bridge.

(3) Subject Sand Volume

Though, as mentioned in the foregoing, check-up of sediment discharge from the mountain area is planned to be performed by means of the group of the Sabo dams and the sand arresting basin. However, according to the construction schedule, construction of the group of the Sabo dams is not completed yet at the time of completion of the river improvement project including the sand arresting basin, and as the result, there is a shortage in the sediment regulation volume.

Therefore, the design sediment discharge are established in two ways ----- 1) period after completion of the Sabo facilities, and 2) temporary period during construction.

+ Subject Earth Volume after Completion of the Sabo Facilities

  Sediment discharge, at the time of flood, to the fan-shaped delta upon completion of the Sabo facilities, is the total discharge, 1,849,000 cu. m. from which the regulated volume by the group of Sabo dams, 1,014,000 cu. m., and sediment discharge flowing to the down-stream river channel, 144,000 cu. m., are deducted. Also, annual mean sediment discharge, after completion, is the volume equivalent to the annual mean sediment discharge before the completion, 328,000 cu. m./year from which the regulated volume by the group of Sabo dams, 24,000 cu. m./year, and sediment discharge flowing to the down-stream river channel, 30,000 cu. m./year, are deducted.

Therefore, the subject sediment discharge upon completion of the Sabo facilities is as shown below.

Heaviest Flood :	691,000 cu. m.
Annual Mean :	274,000 cu. m./year

+ Subject Earth Volume during Temporary Period during Construction

The construction schedule of the Project specifies the construction period of five (5) and fifteen (15) years for the river improvement and the group of Sabo dams, respectively. Therefore, during ten (10) years after completion of the proposed river improvement works, the sediment discharge regulation capacity at heavy floods falls short of the design discharge, 1,014,000 cu. m. So, under heavy floods, sediment discharge beyond the above-mentioned regulation capacity would be carried down to the fan-shaped delta zone. Therefore, during the temporary period --- from the completion of the river improvement works to the completion of the group of Sabo dams --- a temporary reserve is set up in the capacity of the sand arresting basin.

The subject sediment discharge in heavy floods during the above-mentioned temporary period is determined in terms of the sediment discharge from the mountain area at the time of completion of the river improvement works. At this stage, sediment discharge regulation of the Sabo dams at the time of floods is 499,000 cu. m., and this is a short of 515,000 cu. m. against the design discharge regulation of 1,014,000 cu. m.

As an aid to ensure the safety, it is desirable to establish a room for a storage of 515,000 cu. m., however, 50% of 515,000 cu. m. is secured for the relief taking such facts as frequency of occurence of heavy floods is relatively low, and the period in question is limited to ten (10) years only, into consideration.

Also, as annual mean sediment discharge during the same period is considered in storage in the Sabo dams to be constructed in sequence, it is not considered here as the subject sediment discharge. Accordingly, the subject sediment discharge during the temporary period is established as below.

Heaviest Flood :	949,000 cu. m.
Annual Mean :	0 cu. m.

2. Alignment

The alignment of the dike will be according to the draft of B.P.W., and the existing dike 2.7km up the right band of the West Mancatian bridge will be extended 900m upstream to be attached to the mountain. The downstream from the bridge will be according to the B.P.W. proposed dike alignment, which will be connected to the alignment of river course in the midstream in the neighborhood of STA16. As the left-bank dike is located at a higher ground than the right bank, no dike is almost necessary. Therefore, a small dike with a height of 0.5m and a width of 6m concurrently used for the maintenance road will be planned.

On the left bank, this alignment will be of a reduced shape of the draft of B.P.W., no continuous levee is adopted, but an open levee, for the river of mountainous region. This small dike will extend 7.4Km from the pivot of a folding fan of STA 23 + 350m to the downstream river course. There will be constructed an open levee at three places of STA 21 + 150, STA 18 + 100 and STA 21 + 150 in order to introduce valley water from the left bank.

There are several houses in the area enclosed by both the Mancatian bridges, so that a ring dike will be planned for protecting these properties.

3. Longitudinal profile and cross-section

In the case of the longitudinal alignment, the present bed slope of 1/70 will be used, without change, for 2.1km from STA 26 + 400 to STA 24 + 300, and the downstream from STA 24 + 300 will be made an excavated river course. Taking into consideration the river bed fixing and the sand arresting basin which utilizes a broad river width of 700m in the neighborhood of STA 20, the slope will be determined 1/70 and that down the Mancatian bridge of the downstream, 1/90, then the river course will be connected to the downstream one.

The river width of the upstream will be determined from 200 to 100 in the gorge, 650m in the sand arresting basin, 75 to 110m at the East and West Mancatian bridges in the river bed fixing, then 200m after confluence; thus, it will be 120m in the river course of STA 15 + 950 of the downstream.

4. Facility plan

The sand arresting basin shall be divided into the following areas. Individual functions shall be distinguished and then the facility plan which meets the respective requirements shall be established. Selection of performance methods and the facility construction are given in 5-3.

1) Valley .... The dam point - STA.24+300

2) Flow training stretch .... STA.24 + 300 - STA.20 + 650

3) Sand arresting basin.. STA.20 + 650 - STA.19 + 550

4) Diversion and confluence .... STA.19 + 550 - upstream end of the channel

1) Valley area

This valley area will be the 2.2 km long levee utilizing the natural valley from the 1-A dam point to sta.24+300.

The both cliffs of the valley have sharp slopes and are towering. Their top of slope is brittle, the traces of collapse plagued by high water and weathering seen at many places.

The objectives of planning the facility in this section are:

To restrict the further progress of the cliffs' collapse, and

To prevent the passing downstream of the sand produced due to the side erosion of the cliffs and at the river bed, in this section.

In this way the facility plan shall be established as follows:

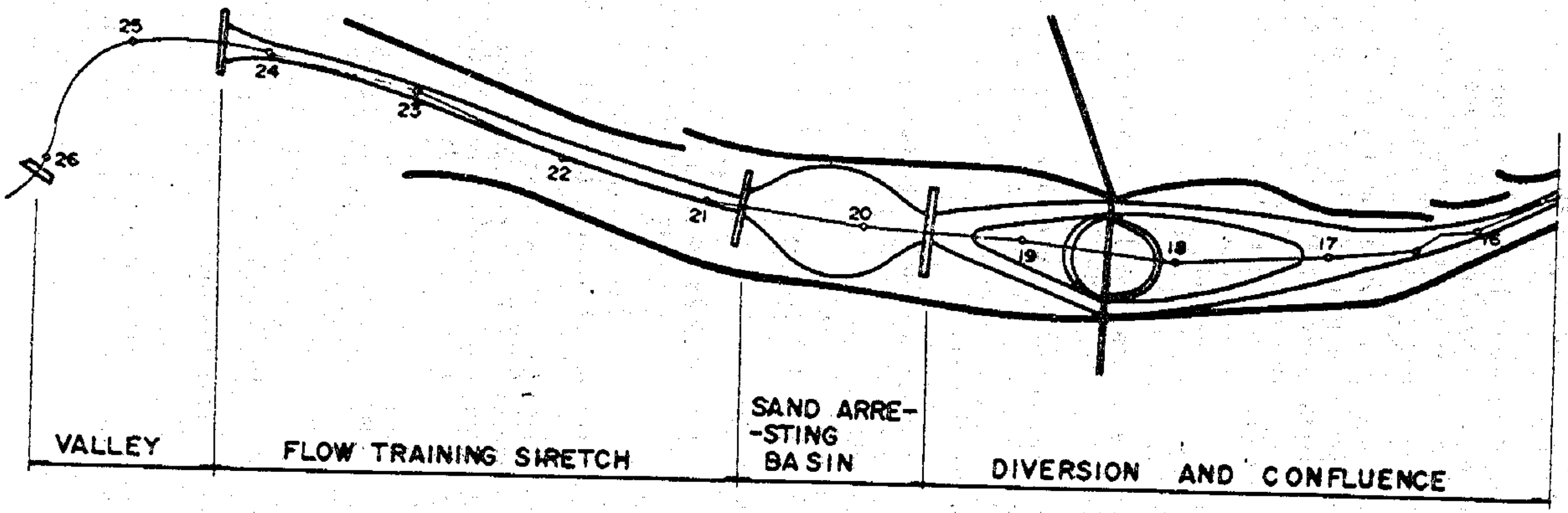


FIGURE Ⅲ-20 FACILITY PLAN SHOWING DEVIDED AREAS

(1) Groyne work

Groyne shall be provided at the base of the cliff slope in this section for the purposes of reducing the impacts caused by streams and runoff sediment at flooding and preventing the side erosion of the cliffs through the effect of sedimentation near the groynes.

The field reconnaissance has shown that some of the sediment effluence from the mountain contain gravels of the sizes larger than 10 cm. Considering this fact, the Groyne Type I shall be employed as the construction of groyne capable of withstanding them. The Groyne Type I will be L = 30 m at 100 m pitches.

(2) Ground-sel

Ground-sel is to prevent the sand produced at flooding in this section from passing downstream.

The ground-sel shall be provided with a waterway. It can be considered that the discharge which the waterway will handle shall be, at the most, Q = 520 m^3/sec. of the ten-year frequency at which small and medium high water occur.

Where Q = 520 m^3/sec., the cross section of the waterway shall be H = 2.50 m and B = 70.0 m.

Wet masonry shall be adopted as examined in 4-2-5, which will cover the sections of STA.26 + 400, STA.25 + 700 and STA.24 + 300.

2) Flow training stretch

Flow training stretch shall be allowed to run a total of 3,650 m over STA.24 - 300 to STA.20 + 650 where there is almost no large-size runoff sediment unlike in the valley.

The main purposes of the facility plan for this section are

To prevent, by fixing the excavated channel course, the disturbance of the stream couse at flooding and allow the design high-water discharge Q = 900 m^3/sec. to safely flow downstream,

To prevent the side erosion of the cliffs near STA.24 + 300 to STA.23 + 400,

To serve as a high-water channel revetment and maintenance road, and

To drain the landside water.

(1) Groyne work

(STA.24 + 300 to STA.23 + 400)

For the purposes of preventing the side erosion of the cliffs and stabilizing the excavated channel, a groyne shall be provided for the full breadth of the high-water channel. The groyne shall be of the pile dike with its reasonable pitch, approximately 1.5 times of the river breadth, on 100 m, which has resulted from the views on the meandering excavated channel and the sedimendation.

(STA.23 + 400 to STA.20 + 900)

Agglomerate is exposed on exists near the ground surfaces of the levees on the left and right of excavated channel. From this fact, the groyne to protect the levees is not required at the present time, but that to stabilize the excavated channel shall be provided. The required groyne will be of the pile dike having L = 40 m at 100 m pitches.

(2) Embankment of Levee and Revetment

1. Left levee

Filling will be performed to form a maintenance road having a crown width of 6.0 m, hight of 0.5 m and slope gradient of 1:30. To protect both the landside and riverside from gully erosion. Sod facing shall be carried out on these sides.

2. Right levee

The dikes under the B.P.W. plan have been completed from the downstream to STA.22 + 100. In order that high water not be allowed to flow into Porac when the excavated channel has been disturbed at flooding, the section of STA.22 + 100 to STA.23 + 100 will be extended in length using the type of bank established by the B.P.W.

3. Open levee

For the purpose of draining the landside water, an open bank shall be provided at one place on the left bank side near STA.21 of this section.

3) Sand Arresting Basin

The section from STA.20 + 650 to STA.19 + 550, totalling 1,100 m in length, has the largest cross-sectional area over the average river breadth of approximately 800 m and along the entire length. Utilizing the large river breadth, this section is chiefly intended to control the design sediment discharge Q = 835,000 m^3.

(1) Shape of sand arresting basin

The sand arresting basin shall have a size and capacity that can control the design sediment discharge, Q = 835,000 m^3. Its plan

(2) Ground-sel

The sandarresting basin is of the excavated channel type so as to control the design sediment discharge V = 835,000 m^3. For this reason, both the inlet and outlet of the sand arresting basin shall be protected in the river bed to stabilize the river bed.

Ground-sel shall be composed of concrete. The waterway, subject to the middle flood discharge Q = 520 m^3/sec., shall have a cross section consisting of B = 70 m and H = 2.5 m. The planned wing extension from the protected river bed shall be L = 150 m both for the outlet and inlet, subject to the design high water discharge Q = 900 m^3/sec.

(3) Groyne work

To assure the alignment of the planned sand arresting basin, pile-dike type groyne shall be provided on the left and right banks at STA.20 to STA.19 + 700 near the outlet of the sand arresting basin, the Groyne being 50 m long, at the pitch of 100 m on the left levee side and 40 m on the right levee side which shall become the concave side of river bent.

(4) Gabion cylinder

For the purposes of stabilizing the inlet and outlet of the sand arresting basin and safeguarding the protected river bed, the slope face of the excavated channels at the inlet and outlet of the sand arresting basin shall be provided width gabion cylinders.

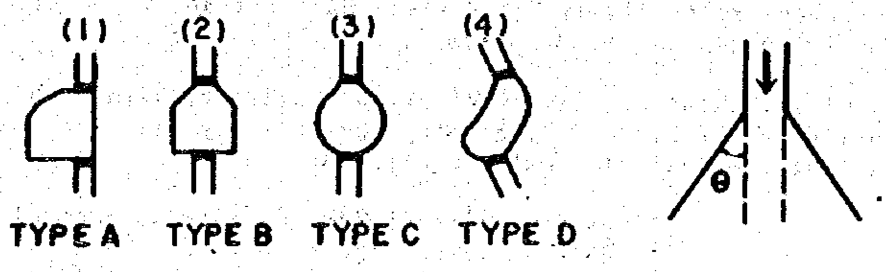
(5) Embankment and maintenance road

The levee under the B.P.W. plan has already been constructed on the right bank side. Therefore this plan desires to utilize the completed levee in its present condition, but the maintenance road having a crown width of 6.0 m shall be provided on the left levee side.

(6) Temporary River Bed

During the period, after completion of the river improvement works, and before the completion of the works of the proposed up-stream Sabo dams, there is possibilities of occurence of sediment discharge above the allowable sediment discharge of 835,000 cu. m. To regulate the above-mentioned excess sediment discharge, the bed of the sand arresting basin is further excavated temporarily by about 50 cm from the design level to secure 949,000 cu. m. in the entire sand arresting basin.

shape will be as illustrated below.



Judging specifically which shape is the best is difficult. However, as sediment gradually moves upstream from near the inlet if enlarging the river breadth drastically at the upstream inlet, the effect of sand arresting basin shall be reduced and in turn adversely affecting the upstream side.

Usually the past records indicate that the best is at angles of 30° to 60°. For this sand arresting basin, adoption of No.3 type is most reasonable with regard to the plan shape of the river and the downstream and upstream formation of the sand arresting basin, and shall be approximately 40°.

Also, in determining the magnitude of the sand arresting basin:

+ avoid exposed rocks as seen in the up-stream in the Near STA 21
+ hydraulically smooth connection in training dike --- sand arresting basin --- branch and junction
+ taking safety of embankment into consideration, a sufficient space may have to be secured between sand arresting basin and embankment
+ the maximum depth of excavation is fixed at 5 meters as the foundation of the proposed sand arresting basin is a fragile sandy soil, and an excessive excavation is not suitable when steadiness of the excavation wall is considered.

Taking all the above-mentioned into consideration, a pocket as big as possible was planned.

4) Diversion and confluence

The diversion and confluence area runs a total length of 3,600 m from STA. 19 + 550, the outlet of sand arresting basin to the termination point of the channel. Near STA.18 + 450 Porac Anjeles road runs across the river, with 75 m and 110 m bridges on the east and west levees respectively. The river has left and right diversions which confluent at the downstream.

On the shoal are living Macatian residents, with such present situation as background, designing the facility plan for this section will list the following main requirements.

- Leading safely to the downstream channel the the high water discharge flowing down from the sand arresting basin.
- Protection Porac residents' living on the shoal from high water.
- Protecting the bottlenecks of the both bridges on the left and right banks of Porac Anjeles road.
- Protecting the levees
- Draining the landside water
- Providing a fuse system against unusual high water.

(1) Excavated channel

The following three alternatives can be considered as the method of operating the shoal-diverted channels.

a. To allow the design high water discharge, $Q = 900\ m^3/sec.$, to flow downstream through the diverted channels.

b. To use the right channel at the normal time to handle small and medium high water, but the both channels at the design high water time to let the high water flow downstream.

c. In contrary to b, above, to use the left channel at the normal time, while the right channel for emergency use.

For Alternative a. it is risky and undersirable to think that calculated water discharge shall be diverted at the design high water. Alternatives b. and c. have introduced the concept of normal and emergency use. The problem will remain in which to use at the normal time.

The past flood trace data indicate that the direction of flown at high water is mainly on the left levee. Thinking together with not allowing high water to flow into Porac side, the present plan shall designate the left channel for use at the normal time and the right one for emergency use.

The inlet of the right channel shall be provided with a wet masonry type overflow weir to distribute the water discharge at the rate of $Q_1 = 520\ m^3/sec.$ for the left channel and $Q_2 = 380\ m^3/sec.$ for the right channel.

The downstream at the shoal is the confluence of diverted channels, which leads to the main channel. In the case of joining the left and right channels together, it is commonly recognized that the angle formed by the direction of flow and the channel is preferably less than 15° hydraulically. Therefore the present plan has determined the line of slope of the left bank as $\theta = 15°$ and that of right one as $\theta = 7°$, with consideration given to the present topographical condition.

(2) Ring levee

For the purpose of protecting Porac residents living on the shoal from high water, ring levee having a crown width of 6.0 m and a slope gradient of 1:3 shall be provided. The ring dikes shall consist of the one at the upstream and the other at the downstream between which Porac Anjeles road is to be located.

1. Upstream ring levee

This dike will be H = 2.0 m in top and 750 m in length. The levee is an important facility to protect the residents on the shoal from high water when the upstream channel is disturbed. Therefore, the ring levee slope face on the river side shall be lined with wet masonry and that on the shoal side sodded with wild grass.

2. Downstream ring levee

This levee shall be H = 3.0 m in top and L = 580 m. The dike has less significance than the upstream levee, but considering the uncertainty in the direction of flow at high water time, the levee shall have its river side provided with gabion cylinders, and its shoal side sodded with wild grass.

3. Groyne work

Groyne work for these sections shall be of pile dike type infended to reduce the impact due to water flow and protect the ring levees through the effect of sedimentation near the groyne. The groyne shall be provided at the concave side of a bent. It shall be 40 m long and at a pitch of 50 m to 100 m. The arrangement of groynes shall be presented in the attached drawings.

4. Garion cylinder

45 cm-diameter gabion cylinders shall be provided on the slope face of excavated channel at the concave side of a bent, to stabilize the channel.

Gabion cylinders shall be arranged as follows:

Left levee side:	Sta.18 + 950 - Sta.19 + 730
	Sta.15 + 850 - Sta.16 + 160
Right levee side:	Sta.19 + 240 - Sta.19 + 790
	Sta.15 + 930 - Sta.16 + 380

5. Revetment and maintenance road

Levee shall be 6.0 m in crown width and at a slope gradient of 1:3. For the purpose of preventing gully erosion, the face of river-side slope shall be sodded with wild grass. Maintenance road shall be 6.0 m in crown width, equivalent to that of the levee.

Left levee

(STA.19 + 550 to Porac Anjeles road)

This section of the levee shall be approximately 1.5 m high, protected with wet masonry.

(Porac Anjeles road to STA.16 + 850)

This shall be the maintenance road which also shall play a role as a open bank. The face of dike shall be sodded.

(STA.16 + 850 to STA.16 + 150)

Approximately 2 m in top, protected with wet masonry.

Right levee

(Confluence from Porac Anjeles road)

The levee and its revetment have already been completed under the B.P.W. plan, and therefore, this levee shall be utilized for the present plan without any improvement.

(STA.18 + 280 to STA.17 + 150)

Approximately 2 m in top, protected with wet masonry.

(STA.17 + 150 to channel's upstream end)

A maintenance road of 50 cm infill height, the face of its slope sodded.

6. Ground-sel

For the purpose of stabilizing the piers of the bridges over the channels on the left and right sides of Porac Angeles road, wet masonry type protection shall be performed at the left and right channels at STA.18 + 280, and, to stabilize the river bed at the termination end of sand arresting basin, similar protection shall be performed.

7. Retaining wall

For the purpose of preventing the abutments on the both levees from collupse, retaining walls shall be provided at the bottlenecks of the left and right banks of Porac Anjeles road. The retaining walls shall be 0.5 m wide in crown, 0.1:1 gradient in front face, 0.5: in rear face, 4.7 m in height. They shall be of gravity type.

New retaining walls of 100 m long shall be provided against the both levees of the left side channel, the existing 50 m long retaining wall on the right side levee of the right side channel shall be further extended by 50 m, and that (30 m long) on the left side levee shall be further extended by 70 m.

8. Open levee

(Left levee near STA.18)

This open levee shall be intended to drain the landside water.

(Left levee at STA.16 + 150)

An open levee, combined with fuse facility for dewatering the landside and unusual high water. The shape of the open levee is shown in the attached drawings.

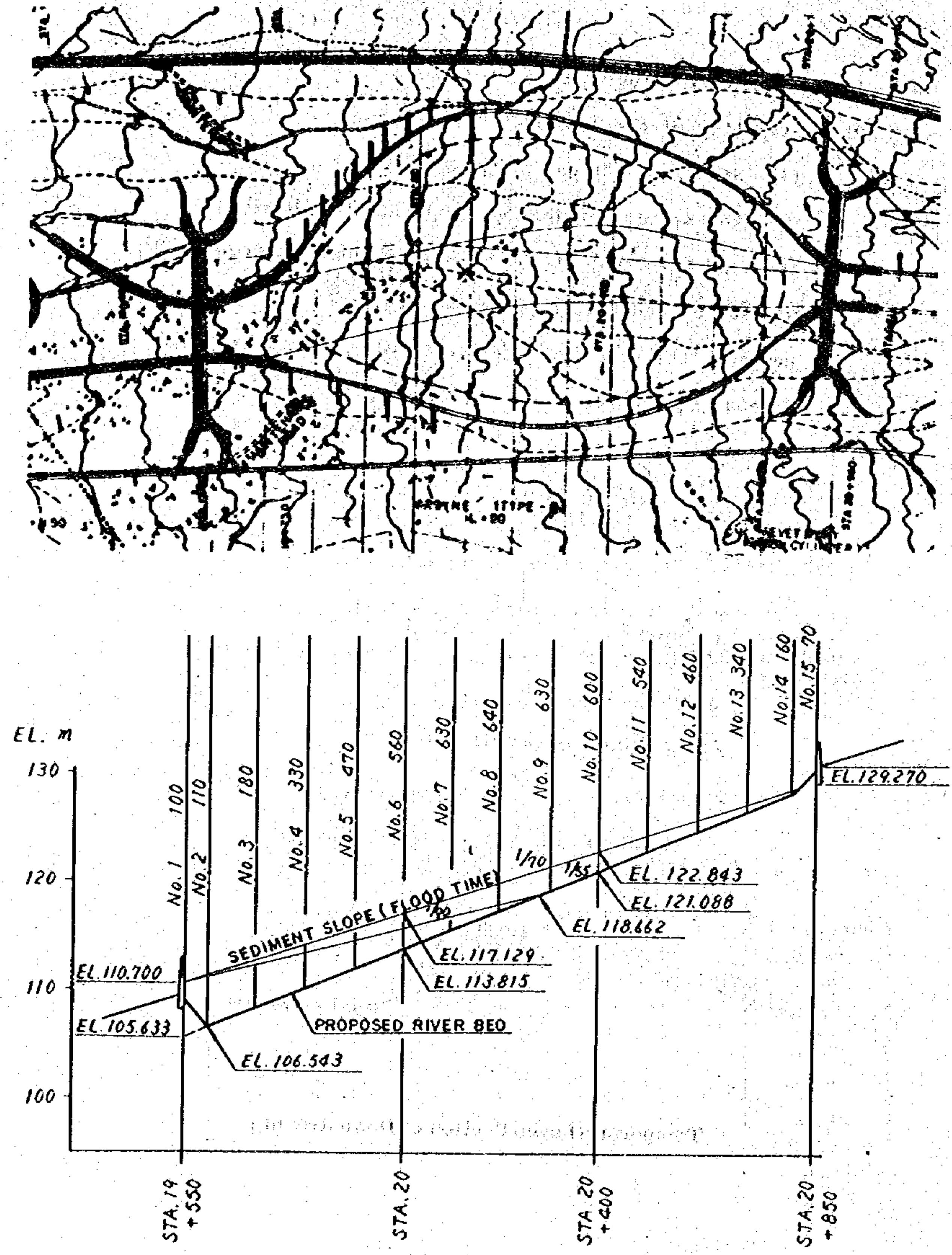
5. Check-up of Sand Storage Capacity

Since the silt gradient of the sand basin is governed by such valuables as flow regime, grain size of sediment, etc., it is rather difficult to theoretically determine silt gradient of sand arresting basin. At location the fan-shaped delta where a steady sedimentation is observed, the gradient is generally in the range as shown below.

$$i = 1/60 - 1/80$$

Accordingly, silt gradient of say, $i = 1/70$ is considering, the silt volume of the sand basin would become about 818,000 cu. m. This is enough to secure the required sediment storage volume. Also, the silt volume during the temporary period is 1,025,000 cu. m., and this satisfies the required design value.

FIGURE III - 21 SAND ARRESIING BASIN



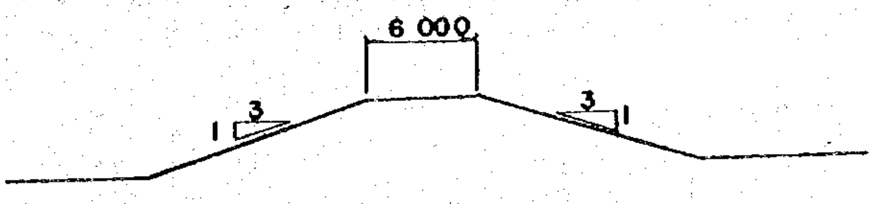
3-5 Levee and Other Structures

1. Stability of the Levee

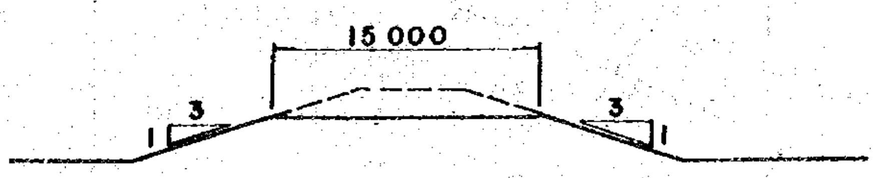
(1) Standard section:

The section of the levee to be newly built will be the same with that of the existing levee; the crown width will be 6m and the slope gradient, 3 : 1. However, the most downward levee will be a temporary one without freeboard.

FIGURE III-22



Levee Section of General Area

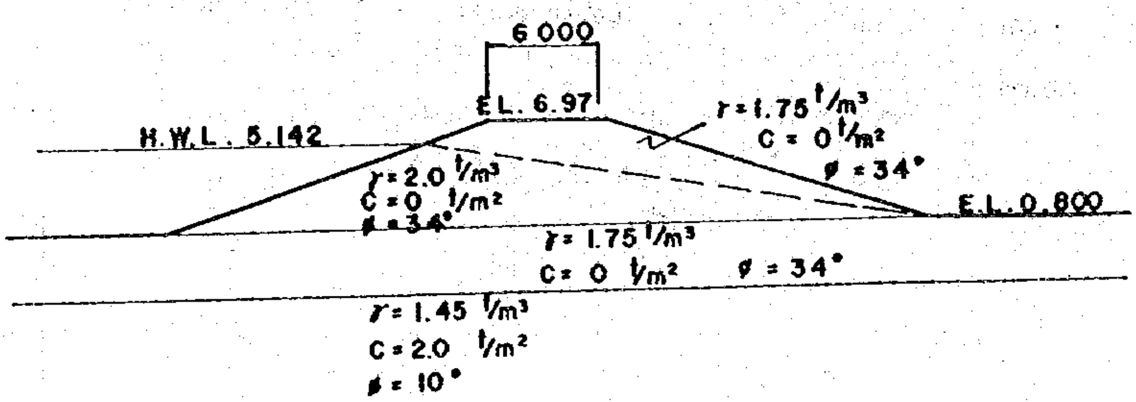


Temporary Levee Section of Downstream

(2) Stability for sliding of the face of slope:

i) Examining conditions:

FIGURE III-23



ii) Calculation of stability:

The results of calculation of circular are sliding are shown in Fig. IV-21-and -2. The following table gives the minimum safety factors under various conditions.

The minimum safety factor for sliding of the face of slope (Section of STA 3)

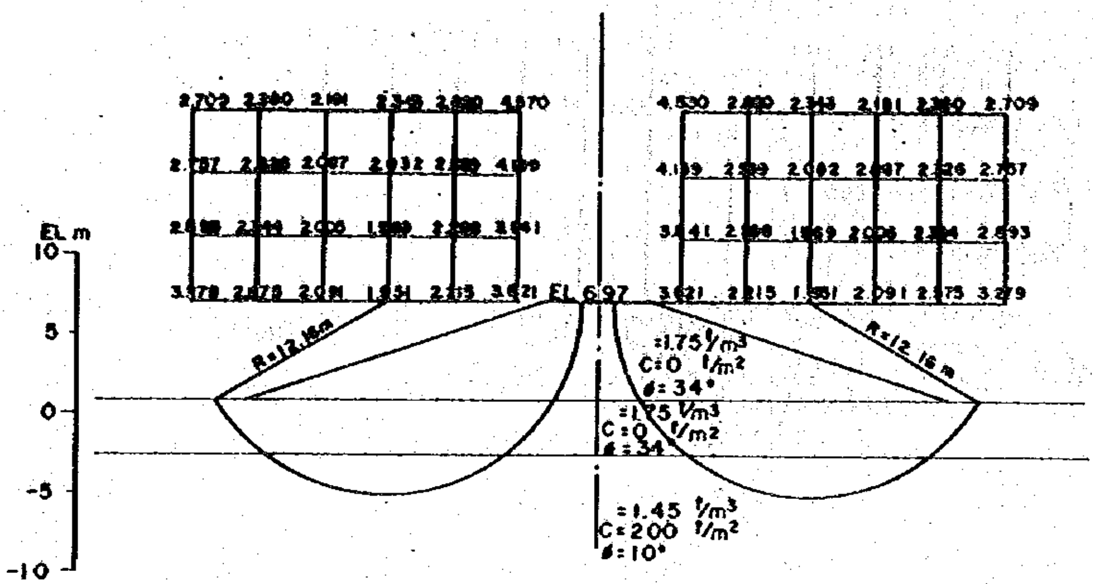
	Channel Condition	River-side Slope	Land-Side Slope
Normal Time	No Water	1.93	1.93
	High Water	2.03	1.15
Earthquake	No Water	1.15	1.15
	High Water	1.00	0.66

According to the results of calculation, there is a problem concerning the stability at the time of earthquake. However, in view of the present situation of the Pasig Potrero river, in whose surroundings of river course are few properties, it is advisable to cope with possible danger due to earthquake by maintenance and repairs.

FIGURE III-24-1

STATION-3 SECTION

(K-0)



STATION-3 SECTION

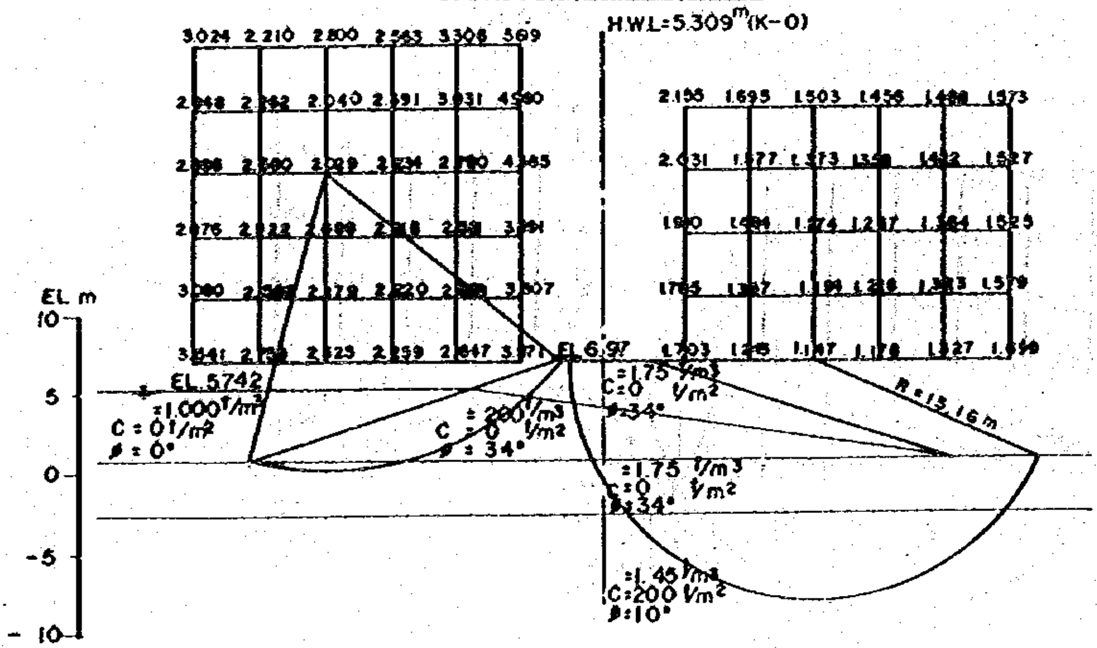
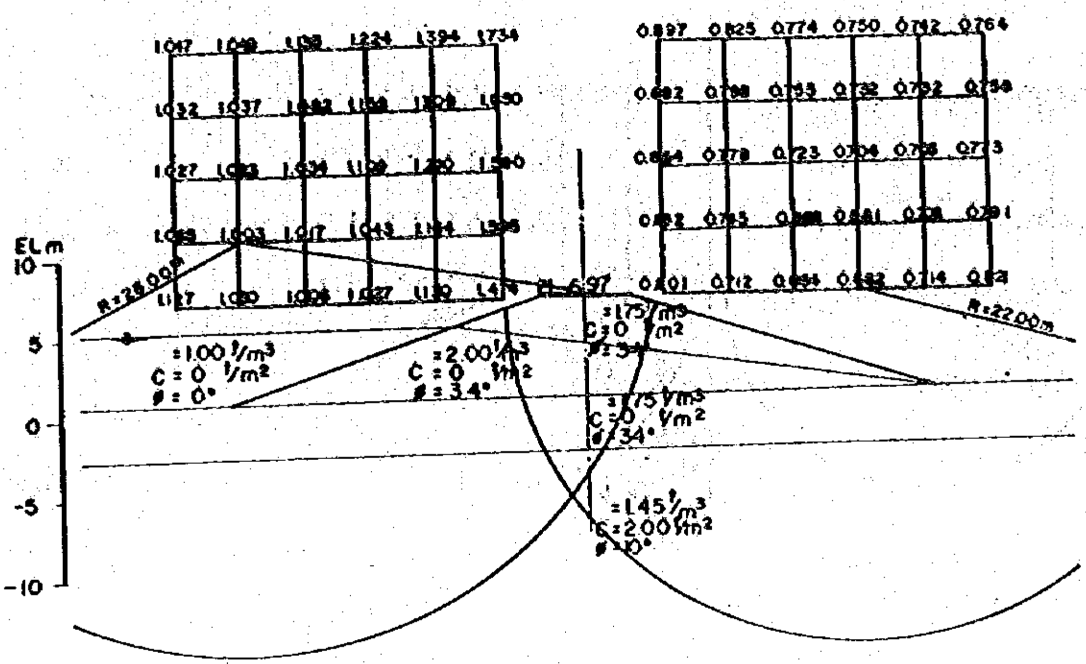


FIGURE III-24-2

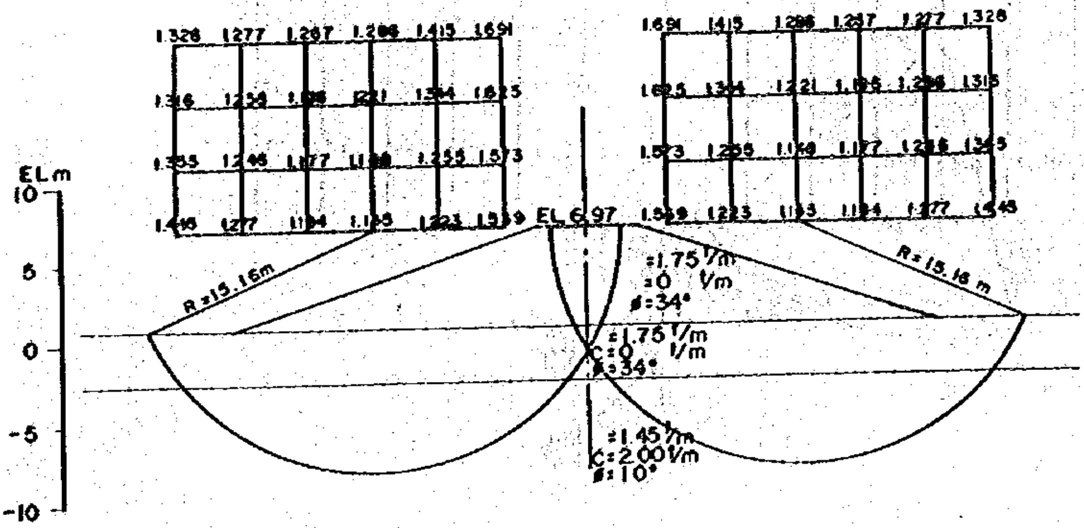
STATION-3 SECTION

HWL 5309^m(K=0.12)



STATION-3 SECTION

(K=0.12)



(3) Examination of piping:

(i) Examination conditions:

Section to be examined: The same section as that for the calculation of stability (Refer to para. 3-5, 1, (2).)

Soil condition: Unit weight of soil constituents (Gs) = 2.57 kg/cm^3

Void ratio (e) = 0.54

Coefficient of permeability (k) = 1.0 x 10^{-3}

(ii) Examination by critical hydraulic gradient

where there is a difference in the stage of water in ground and the water is in a flowing state, the soil constituents undergo infiltration pressure along the stream line. Then, if the hydraulic gradient caused by the difference in stage increases more than a certain limit, the soil constituents start moving. The hydraulic gradient at the time the soil constituents start moving is called the critical hydraulic gradient, which can be calculated by the following formula.

$$Ic = \frac{Gs - 1}{1 + e}$$

Where Ic: Critical hydraulic gradient

Gs: Unit weight of soil constituents (=2.57t/m^3)

e: Void ratio of soil (=0.53)

$$Ic = \frac{2.57 - 1}{1 + 0.53} = 1.026$$

While, the hydraulic gradient in the dike can be calculated by the following formula.

$$I = \frac{H1 - H2}{L}$$

Where Ic: Hydraulic gradient

H1: Stage outside the dike (EL. 5.142m)

H2: Stage inside the dike (EL. 0.8m)

L : Horizontal length of the infiltration section (29.994m)

$$I = \frac{5.142 - 0.800}{29.994} = 0.144$$

As the hydraulic gradient (I) is considerably lower in value than the critical hydraulic gradient (Ic), safety is ensured against piping.

(iii) Examination by Justin's critical velocity:

Paying attention to the energy of velocity of flow of the water flowing inside the ground, this method is based on the thinking that when the velocity of flow exceeds a certain limit, soil constituents are caused to move and the piping phenomenon occurs. The then velocity of flow is called critical velocity which is determined depending on the nature of soil.

Justin provides the various particle diameters of soil constituents with the respective critical velocities of flow as shown in the following table.

TABLE III-9 CRITICAL FLOW VELOCITY FOR BAD GRADING DISTRIBUTION

Sample No.	Grain size (mm)	Critical flow velocity (cm/sec.)
1	4.0 ~ 4.8	0.200
2	2.8 ~ 3.4	0.170
3	1.0 ~ 1.2	0.100
4	0.7 ~ 0.85	0.085
5	0.4 ~ 0.7	0.070
6	0.25 ~ 0.5	0.042
7	0.11 ~ 0.25	0.035
8	0.075 ~ 0.11	0.025
9	0.044 ~ 0.075	0.020

While, the infiltration velocity inside the ground can be calculated by the following formula.

$V = K.I$

Where V: Infiltration velocity (cm/sec.)

K: Coefficient of permeability ($=1.0 \times 10^{-3}$)

I: Hydraulic gradient ($= 0.144$)

$$V = 1.0 \times 10^{-3} \times 0.144 = 1.4 \times 10^{-4} \text{ cm/sec}$$

The calculated infiltration velocity is of a small value in terms of order compared with the critical velocity, so that it is safe from piping.

(4) Settlement due to Consolidation:

It is estimated in the light of the results of soil test conducted at the location of the High Way bridge (Data No.1-7) that the silt layer distributed downstream from the point of Sta.4 will consideraly settle after banking.

However, according to the actual measurement of the settlement due to consolidation in the neighborhood of the above point, it is recorded that the settlement due to consolidation resulting from road filling is 66cm, as comparatively small value, for two years. Judging from the trend of decrease in settling velocity, it is considered that this value can be taken as such the consolidation has ended approximately 90%. Seeing that the actual record is as above and that the planned height of dike is almost the same as the height of road filling (load) under construction, it is estimated that the settlement due to consolidation after banking is approximately the measured value mentioned above. Accordingly, it may be determined the extra-banking to compensate for the settlement due to consolidation is approximately 70cm.

2. Revetment

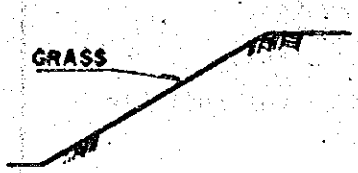
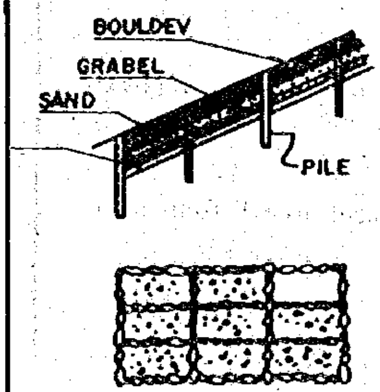
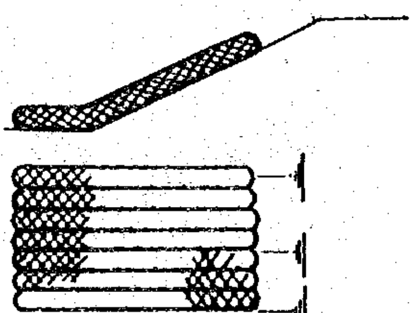
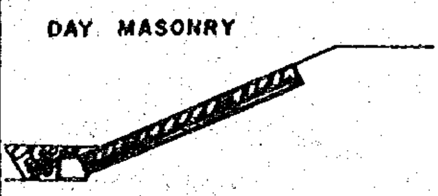
(1) Selection of kinds of work:

The revetment of the face of slope consists of sodding, hurdle work, crating, masonry, concrete stab work, asphalt facing, etc. (See Table IV-6.)

In this repairing project, appropriate ones of them will be used taking into consideration the availability of materials, their durability and strength, workability, and the cost of work and so on.

The materials which are easily available in the area concerned are sand, stone, wood, bamboo, etc. Judging from the fact that the present revetment is of dry pitching, it is reasonable to use stone material.

TABLE III - 10\ THE EXAMPLES OF REVETMENT

TYPE	REFERVCE RICTVRE	EXPLANATION
SODDING CONSTRUCTION		cover the face of the slope by sodding placement above the ordinary water level of the slow-stream section normal velocity of flow between 1 and 25 m./sec. approx.
HURDLE CONSTRUCTION		fabricate fascine into hurdles, and fill with cobbles, boulders and gravels relatively higher roughness, and lower costs suitable for a slow-stream section lack in durability
GABION		fabricate gabions with bamboo and iron wire, fill with cobbles, boulders, etc., and lay on the face of the slope flexible and simple in use low durability against corrosion and impact of floodwood, etc.
STONE - PITCHING CONSTRUCTION	 	stones laid on the face of the slope a. Dry Masonry - application w/o mortar require use of considerbly heavy stones to prevent wash away by floods b. Wet Masonry - application w/ mortar

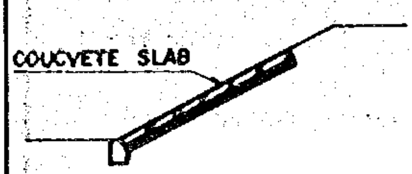
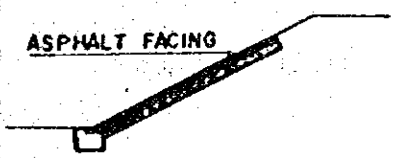
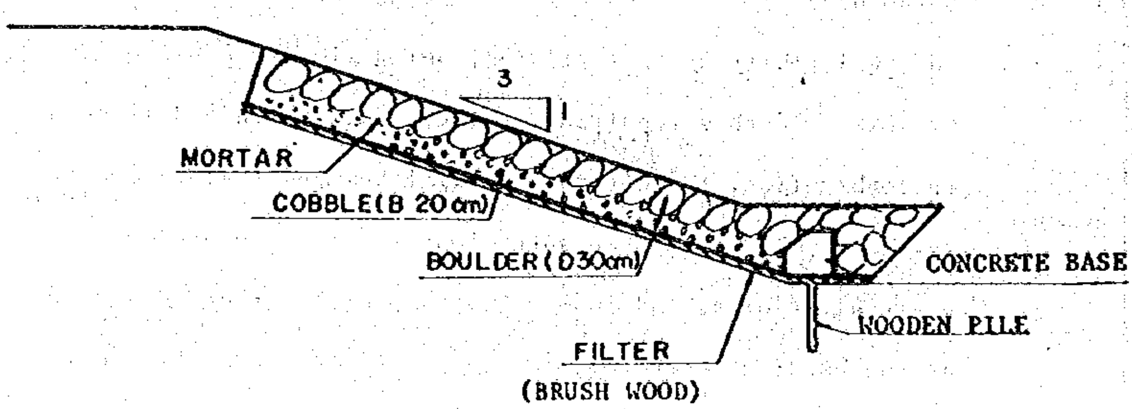
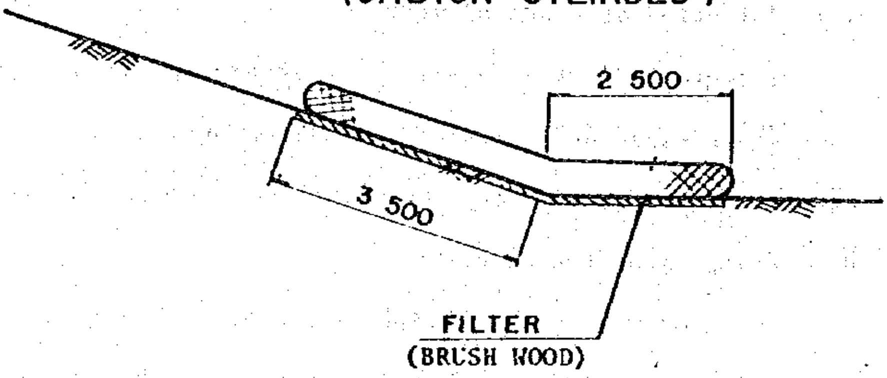
	REFERENCE PICTURE	EXPLANATION
CONCRETE FACING		cover the face of the slope with concrete of 10 to 15 cm. in thickness applied in rapid stream section lacks flexibility, hence, not suitable for a poor foundation
ASPHLT FACING		cover the face of the slope with asphalt compound need well-rolled foundation need a gentle slope, and sound foundation treatment

FIGURE III-25 TYPE OF REVETMENT

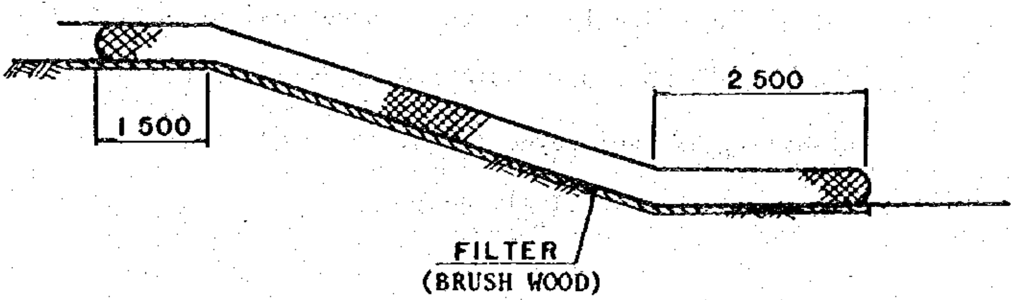
WET MASONRY



LAND SIDE RERVTMENT
(GABION CYLINDED)



LOW WATER RERETMENT (GABION CYLINDER)



Of the revetment using stone, the stongest is the wet masonry. The wire cylinder work is short of durability compared with other kinds of work, but it excels in flexibility and resistivity against erosion, being simple in execution of work. In this repairing project, both of them will be used in combination; the wet masonry will be used in the case of revetment which requires sufficient strength to prevent breaking of dike, and in the case of the low flow revetment or the backing revetment, the gavion cylinder work will be applied.

(2) Standard profile

The type adopted to this Project is shown in FIGURE IV-22.

3. Groyne Work

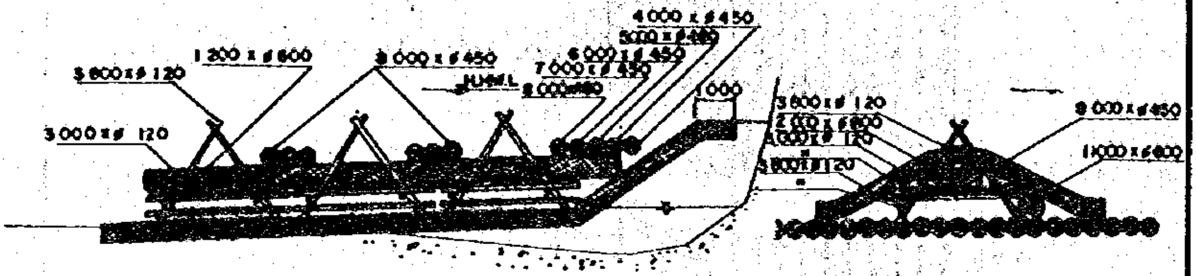
(1) Selection of kinds of work:

The groyne work is a structure accomplished through many years experience after installation on trial, and consists of various types. TABLE IV-7-1 and TABLE IV-7-2 show some examples of the structure.

The groyne is classified into the permeable groyne and the impermeable groyne according to their structures. The permeable groyne is made so that part of flow is capable of being permeated. The velocity of flow is reduced by means of the members of the groyne; as a result, settlement of flowing down sediments is promoted. Accordingly, the resistance against flow is lower than the impermeable groyne, so that the stability of the groyne itself is good, and its maintenance is comparatively easy.

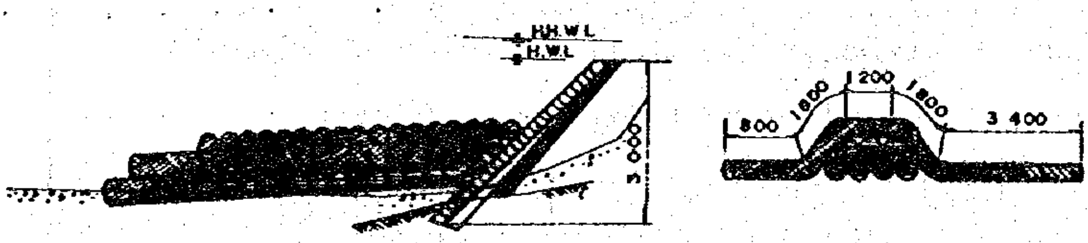
TABLE III-11-1 TYPE OF GROYNE

GROYNE (TYPE 1)



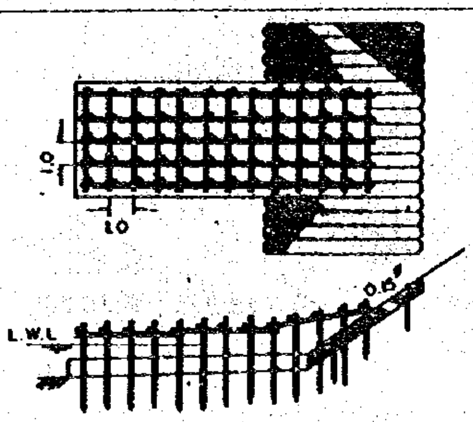
Groyne cylinders placed on a frame fabricated with logs.
Commonly used in rapid-stream sections

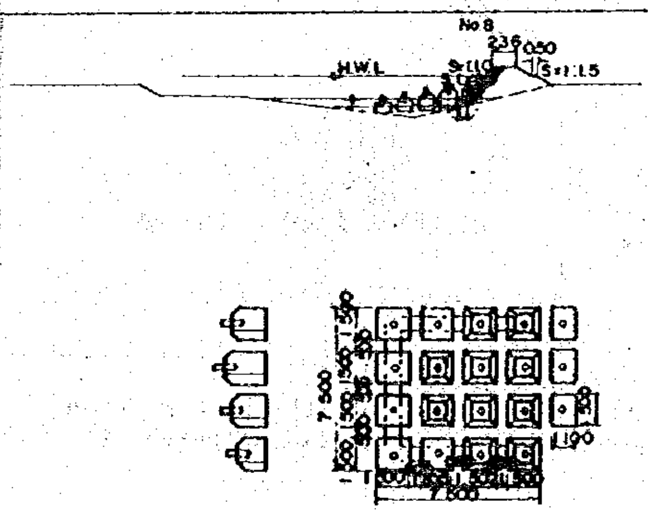
GROYNE (TYPE 2)



Gabions are placed in 2 - 3 layers at right angles with the direction of the flow, and on top of them more gabions are placed as weight in the direction of the flow

TABLE III-11-2 TYPE OF GROYNE

GROYNE (TYPE 3)

Piles are driven in several rows. A representative permeable groyne

GROYNE (TYPE 4)

Concrete blocks are laid in rows.

The impermeable groyne is so constructed that flow is not capable of being permeated; its water splashing effect is remarkable, but the resistance against flow is so high that deep digging is apt to occur in the surroundings of the groyne. Therefore, it is necessary to make the groyne flexible, or to protect the surroundings of the groyne with mattress, rubble-mound, etc.

The Pasig-Potrero river included in this repairing project has a bed consisting of sand; therefore, there is a fear that the surroundings of the groyne may be scoured. Thus, in this repairing project, it is determined to use the permeable groyne excelling in stability, and the skeleton-work and the pile groynes will be selected. The skeleton work is a structure often used for rivers with rapid stream, and has high resistivity to the flow and run-off sediments. The pile dike groyne is a type often seen in rivers with slow stream.

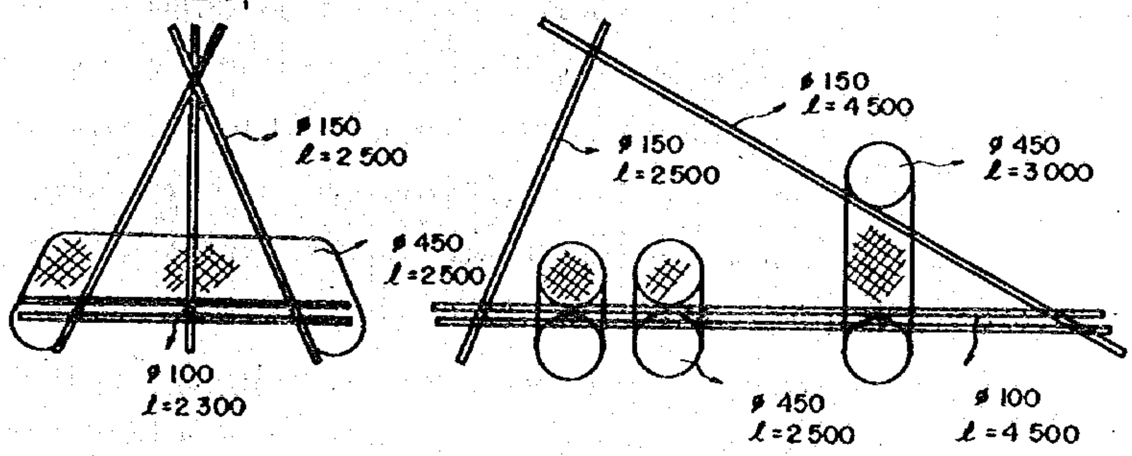
In the case of the Pasig-Potorero river, there is a possibility that run-offs of cobble stone may occur at the time of flood. The skeleton-work groyne will be installed in the section between STA. 27 and STA. 24, and in the downstream from it, the pile groyne will be applied.

(2) Standard profile

The standard profile adopted to this Project is shown in FIGURE IV-23 and FIGURE IV-24.

GROYNE (TYPE A)

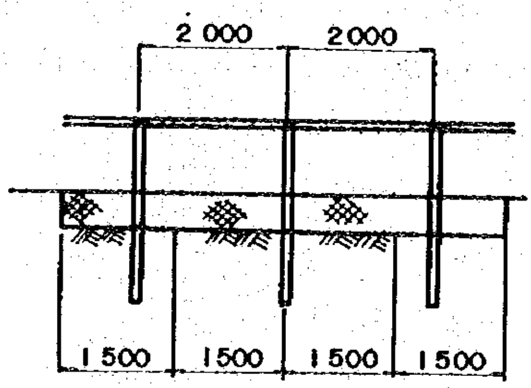
FIGURE III - 26



GROYNE (TYPE B)

FIGURE - III - 27

(SIDE UIEW)

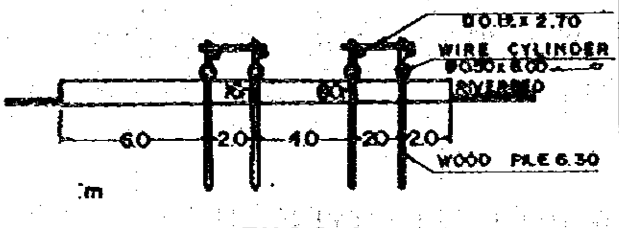
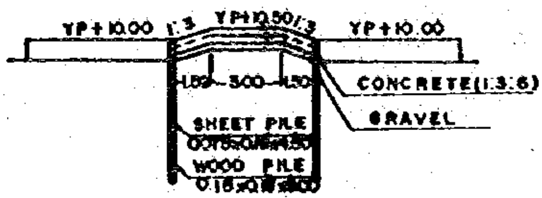
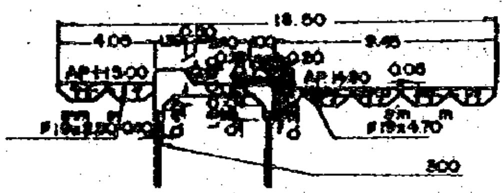


4. Groundsel

(1) Selection of kinds of work:

The groundsel for preventing the settling of river bed includes the mattress, masonry, concrete stab and the like kinds of work.

FIGURE III-28 KINDS OF GROUNDSEL

 TYPE.1	Simple construction by fixing fascine mattresses with wooden piles
 TYPE.2	Place wet masonry over a gravel groundsill. Construct aprons in both up- and down-stream sides
 TYPE.3	Groundsill of concrete construction

In this repairing project, it is intended to use materials which are easily available and adopt a simple method of work, and as a rule, the wet masonry work will be used, but two units to be installed at the ends of

FIGURE III-29 GROUNDSEL (TYPE A) (WET MASONRY)

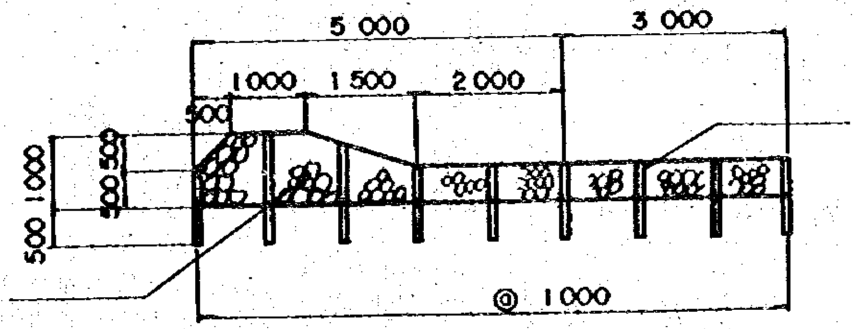
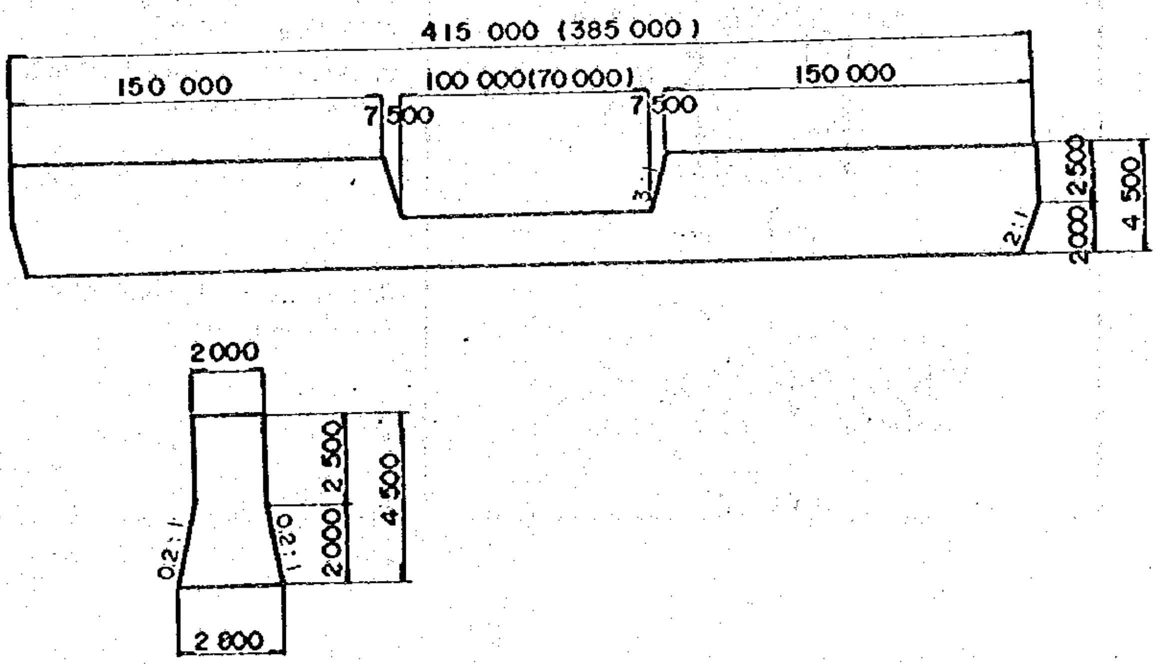


FIGURE III-30 GROUNDSEL (TYPE B) (CONCRETE)



upstream and downstream of the sand arresting basin will be of concrete structure.

(2) Standard profile

The standard profile adopted to this Project is shown in FIGURE IV-26 and FIGURE IV-27.

5. Ring levee

In the case of the ring dike to be installed in the sorroundings of Mancatian village, the crown width will be 6m, the slope gradient, 3 : 1 in the section. Revetment will be applied to the face of slope outside the levee, and the upstream levee will be of the wet mansonry revetment, and the downstream levee will be of the wire cylinder revetment.

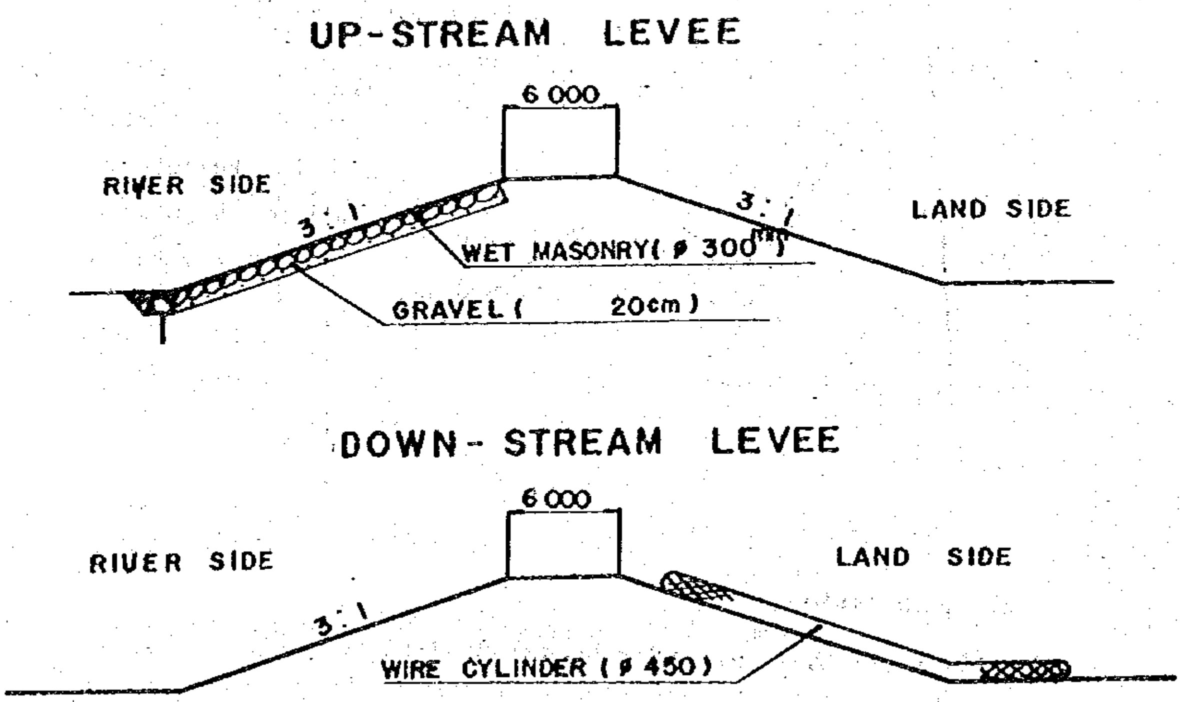


FIGURE III-31 : RING LEVEE

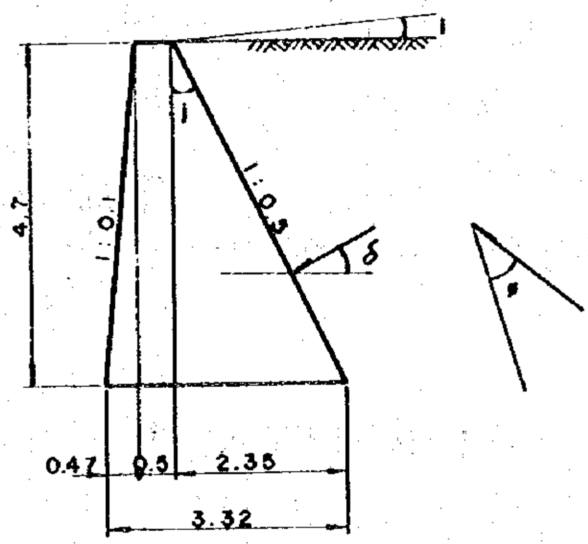
6. Retaining Wall

(1) Standard section:

In the channel water of the point of the Mancatian bridge which is a bottleneck, a concrete gravity type retaining wall will be installed just down the bridge in order to prevent erosion and collapse of the abatment. The shape of the section of the retaining wall will be a trapezoid with a front slope gradient of 0.1 : 1, and with a rear slope gradient of 0.5 : 1.

(2) Stability Computation

1) Conditions



$j = 26.5^{\circ}$
$\phi = 34^{\circ}$
$\delta = 17^{\circ}$
unit wt. of concrete
= 2.3 kg/cu. m.
unit wt. of soil
= 1.75 kg/cu. m.

2) Computation

i) Earth Pressure under Normal Conditions

$$P_A = 1/2\,\gamma H^2 \frac{K_A}{\cos j \cdot \cos \delta}$$

$$K_A = \frac{\cos(\phi - j)\ \cos j}{\cos j \cdot \cos(j+\delta)\cdot\left\{1 + \sqrt{\frac{\sin(\phi + j)\cdot \sin \phi}{\cos j \cdot \cos(j+\delta)}}\right\}^2}$$

ii) Earth Pressure during Earthquake

$$P_{AE} = 1/2\gamma H^2 \cdot (1 - k_1) \cdot \frac{K_{AE}}{\cos j \cdot \cos \delta}$$

$$K_{AE} = \frac{\cos^2(j + \quad + \phi) \quad \cos\delta}{\cos\theta \cdot \cos j \cdot \cos(j \pm \theta + \delta) \cdot \left\{1 + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi - \theta)}{\cos(j + \theta + \delta) \cdot \cos j}}\right\}^2}$$

provided

Seismic coefficient, horizontal, $k = 0.12$

" " , vertical, $k_1 = 0.1$

$\tan^{-1}\theta = k/(1 - k_1)$

Values of K_A and K_{AE} are computed as follows:

$K_A = 0.398$

$K_{AE} = 0.538$

Under normal conditions:

TABLE III-12: STABILITY CALCULATION (NORMAL CONDITION)

Items	V(t)	H(t)	x(m)	y(m)	M(t-m)
Concrete, Dead Load (1)	5.08	-	0.313	-	1.590
" " (2)	5.41	-	0.720	-	3.895
" " (3)	12.70	-	1.753	-	22.263
Soil Load	9.66	-	2.537	-	24.509
Soil Pressure	-	8.99	-	-1.567	-14.087
Total:	32.85	8.99			38.168

$$\frac{M}{V} - \frac{L}{2} = 0.498 \qquad \frac{L}{6} = 0.553$$

The resultant force of the load, V and H, rest in the area within the middle one-third of the base of the structure, therefore, no tensile stress is observed in the structure.

With regard to slide:

$\tan 34 \times 32.85 = 22.15 \geqq 8.99$

this satisfies conditions governing slide,

$\tan\phi \times$ (total V) $\times$ (total H)

so the structure is safe from danger of slide.

During Earthquake:

TABLE III-13 STABILITY CALCULATION (EARTHQUAKE)

Items	V(t)	H(t)	x(m)	y(m)	M(t-m)
Concrete, Dead Load (1)	5.08	-	0.313	-	1.590
" " (2)	5.91	-	0.720	-	3.895
" " (3)	12.70	-	1.735	-	22.263
Soil Load	9.66	-	2.537	-	24.509
Seismic Force, Dead Load (1)	-	0.61	-	-1.567	-0.956
" " (2)	-	0.65	-	-2.350	-1.528
" " (3)	-	1.64	-	-1.567	-2.570
Seismic Force, Soil Load	-	1.16	-	-3.133	-3.634
Soil Pressure	-	10.93	-	-1.567	-17.128
Total:	32.85	14.99			26.441

Since $\frac{M}{V}$ = 0.805, obtained as the result of the above-mentioned computation, is smaller than $\frac{L}{2}$ = 1.66, the structure in question is also safe against earthquake.
With regard to slide, it satisfies the conditions governing slide,

$\tan\phi$ x total V = 22.15 is smaller than total H=14.99,

the structure is safe from danger of slide.

3-6 Channel Stability

1) Flow capacity

This river improvement plan is established on the condition that it is possible to flow the discharge of Q = 120m^3/sec. (probability : 1/1.1year) on the low water channel and that it is possible to safely flow the discharge of Q = 900m^3/sec. (probability : 1/80year).

For examining the discharge capacity of the designed river course, the calculation of non-uniform flow is employed to calculate the level of water at each point of place. The results of calculation of the flows in the five cases of Q = 120m^3/sec. (1/1.1year), 400m^3/sec. (1/5year), 520m^3/sec. (1/10year), 900m^3/sec. (1/80year) and 1,100m^3/sec. (1/200year) are as shown in TABLE III-15..

According ot TABLE III-15, the designed crown height of levee has enough freeboard (1.5m) to enable to discharge the designed discharge of Q = 900m^3/sec. Moreover, the freeboard under the gurder of the bridge which is a bottleneck of the discharge capacity of flow is fully secured as shown in the following table.

TABLE III-14 FREE BOARD AT BRIDGE

STA.	Name of Bridge	Free Board
STA. 2 + 330	Bacolor Bridge	2.3 m
STA. 4 + 200	Highway Bridge	3.2 m
STA. 18 + 400	Mancation Bridge	1.9 m

2) Bed Stability

The stability of the designed river bed is examined from the point of view of the longitudinal continuity of sediment transport.

TABLE III-15 | VARID FLOW CALCULATION RESULT (PROPOSED CHANNEL)

STA	Proposed River Bed	The Top of Proposed Levee	Water Level EL. (m)				
			Q = 120	Q = 400	Q = 520	Q = 900	Q = 1.100
0	-2.500	R = 3.50 L = (3.50)	-1.000	0.700	1.300	3.500	3.500
0.7	-2.111	3.78 (3.78)	-0.254	1.355	1.930	3.779	3.913
1	-1.944	5.40 (3.90)	-0.095	1.567	2.130	3.898	4.076
1.8	-1.500	5.64 (5.64)	0.274	1.932	2.465	4.136	4.386
2	-1.389	5.67 (5.68)	0.346	1.983	2.507	4.161	4.419
2.3	0.227	6.628	1.178	2.407	2.874	4.370	4.666
3	0.689	6.65 (6.97)	3.297	4.172	4.399	5.142	5.527
4	1.379	7.32 (7.62)	3.481	4.599	4.918	5.816	6.235
4.2	1.517	10.30 (10.30)	3.538	4.697	5.029	5.942	6.364
5	2.517	8.36 (8.36)	4.351	5.615	5.984	6.855	7.253
6	4.517	10.25 (10.25)	6.365	7.640	8.001	8.746	9.087
7	7.850	13.70 (13.53)	9.327	10.466	10.796	11.508	11.814
8	12.850	18.62 (18.43)	14.201	15.232	15.444	16.116	16.314
9	17.850	23.05 (22.58)	19.309	20.287	20.477	21.024	21.187
10	23.379	28.52 (27.92)	24.692	25.314	25.530	26.090	26.266
11	29.261	33.20 (35.10)	30.540	31.187	30.329	31.695	31.851
12	35.143	39.36 (39.44)	36.469	37.229	37.381	37.855	38.070
13	41.811	45.84 (46.57)	43.041	43.646	43.886	44.331	44.516
14	48.934	53.15 (53.15)	50.217	51.099	51.240	51.647	51.843
14.8	54.668	59.00 (60.80)	55.911	56.691	56.919	57.494	57.764
15	56.668	60.70 (61.50)	57.999	58.494	58.708	59.205	59.380
16	66.668	70.80 (70.60)	67.351	67.961	68.149	68.649	68.882
16.2	68.668	72.50 (72.50)	69.220	69.783	69.983	70.481	70.715
16.4	70.668	73.90 (73.60)	70.979	71.382	71.528	71.950	72.156
17	76.668	81.30 (81.70)	77.013	77.415	77.556	77.945	78.119
17.4	81.112	84.50 (85.30)	81.395	81.685	81.773	82.015	82.122
18	87.779	92.50 (92.30)	88.286	88.845	89.019	89.315	89.458
18.4	93.493	98.70 (98.70)	94.194	94.781	94.989	95.509	95.752
19	102.065	107.72 (105.50)	102.622	103.232	103.401	103.868	104.077

(to be continued)

STA	Proposed River Bed	The Top of Proposed Levee	Water Level EL. (m)				
			Q = 120	Q = 400	Q = 520	Q = 900	Q = 1.100
19.4	107.779	114.66 (112.20)	108.219	108.750	108.929	109.313	109.488
20	115.429	125.19 (121.20)	115.623	115.862	115.945	116.172	116.277
20.4	121.143	132.45 (127.70)	121.330	121.561	121.641	121.860	121.963
21	133.567	142.64 (138.00)	134.321	135.197	135.467	136.062	136.190
21.4	140.234	149.64 (146.50)	140.969	141.855	142.124	142.572	142.809
22	150.234	160.55 (157.90)	150.995	151.905	152.213	153.023	159.849
23	166.425	178.30 (176.60)	167.631	168.709	169.072	169.654	169.926
23.4	172.774	183.77 (183.39)	174.241	174.729	174.893	175.325	175.525
24	182.294	192.11 (194.81)	183.639	184.170	184.300	184.657	184.823
24.4	188.080	198.22 (202.25)	189.203	189.971	190.151	190.618	190.837
25	196.610	216.45 (212.99)	197.275	197.801	197.985	198.320	198.474
25.4	201.360	220.83 (218.35)	202.088	202.479	202.615	202.992	203.142
26	208.800	234.72 (221.62)	209.308	209.748	209.900	210.319	210.515
26.4	213.440		214.192	214.680	214.849	215.290	215.491
27	222.370		223.038	223.573	223.759	224.270	224.508

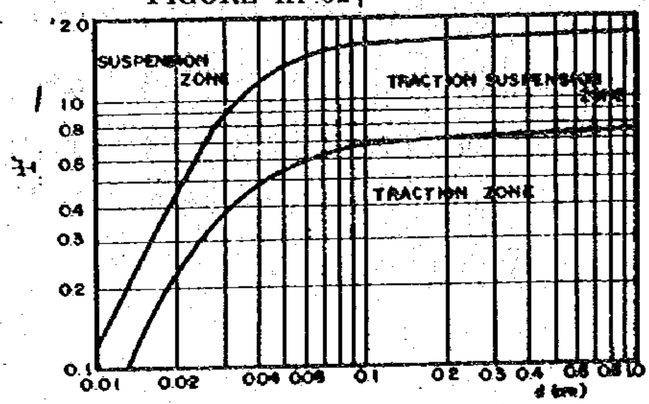
The sediment is roughly classified into bed material load, suspended load and wash load. The bed material load is carried away by rolling or sliding, and the suspended load is carried away in the manner that some find sand on the river bed flows suspended in water. The wash load is quite different from both above, and means that very fine sand independent of the river bed is carried away at the time of flood. There are various kinds of calculation formulas for the bed material load and the suspended load, but strictly speaking, the calculation formulas of sediments are different from one another depending on the characteristics of regime (topography, geology, meandering and stream regime) of the respective rivers; while, there remains a question about whether an actual phenomenon can be perfectly expressed by means of one-dimensional analysis through a sediment formula. However, in this project, the most generally used formula (of bed material load) of Sato, Kikkawa and Ashida, and Lane-Kalinske formula (of suspended load) are used to express actual phenomena.

i) Discrimination of patterns of sediments:

Examining the results of the mechanical analysis of sand on the bed (cf. Fig.III-35-1 - Fig.III-35-5), the grain diameters are different in the upstream and the downstream from the point of approximately 6km as a boundary of the river. (For example, d_{50} = 0.01mm in the neighborhood of 1km, and d_{50} = 0.5mm in the neighborhood of 15km)

Examining the relation between the tractive force ($\Psi = \frac{u_*^2}{Sgd}$; where u_* : friction velocity, S: specific gravity of sand in water, g: acceleration of gravity and d: grain dia.) and the grain diameter, $d < 0.1$mm means the suspended load region. This suggests that the very fine sand accumulated down the neighborhood of 5km contains much of the wash load carried away from the upstream and that variations in bed sand such as the bed material load and the suspended load take place up the neighborhood of 5km. (This agrees with the fact that the influence of the back water of the Gua-Gua River extends to the neighborhood of the High way By-pass bridge at STA. 4+200)

FIGURE-III-32



ii) Calculating expression

As to the calculation of sediment, find the hydraulic quantity by use of the non-uniform flow calculation, then obtain the quantity of sediment by the following formula. For reference, the calculation is carried out by Laursen formula.

a) Sato- Kikkawa-Ashida formula

$$\frac{q_B}{U_*^3 \cdot f(\tau_c/\tau_o)} = 0.623 \left(\frac{1}{40n} \right)^{3.5} \cdots\cdots (n<0.025)$$

$$= 0.623 \quad \text{--------------} (n \geq 0.025)$$

Where q_B : Quantity of bed material load per unit width and unit time

S : Specific gravity of sand in water

g : Acceleration of gravity

U_* : Friction velocity ($= \sqrt{gRI}$)

τ_c : Critical tractive force

τ_o : Tractive force

n : Manning's roughness coefficient

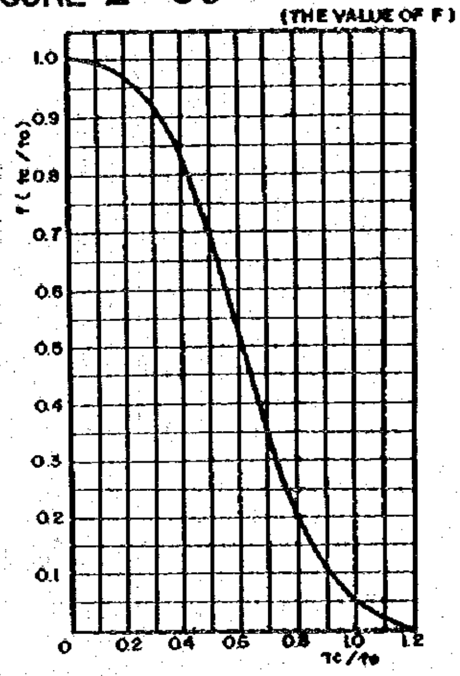
τ_c and f can be found by the following.

TABLE III-16

CRITICAL TRACTIVE FORCE
(BY THE FORMULA IWAGAKI)

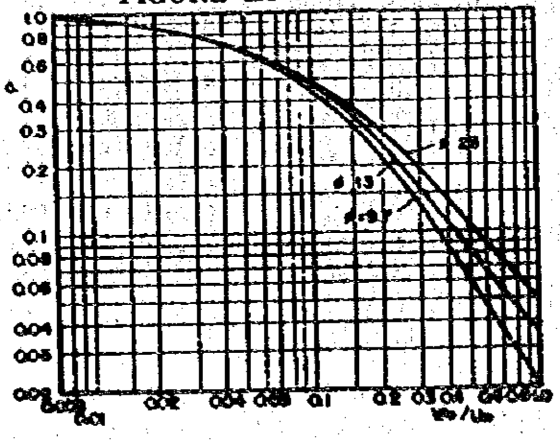
GRAIN SIZE	U_{*c}^2
$d \geq 0.303$ cm	$80.9d$
$0.118 \leq d \leq 0.0303$	$134.6\,d^{31/22}$
$0.0565 \leq d \leq 0.118$	$55.0d$
$0.0065 \leq d \leq 0.0565$	$8.41d^{11/32}$
$d \leq 0.0065$	$226\,d$

FIGURE III-33



b) Lane-Kalinske formula

FIGURE-III-34



$$q_s = q \cdot Co \cdot P$$

$$Co = F(Wo) \cdot 5.55 \cdot P_*^{1.61}$$

$$P_* = 1/2 \cdot \frac{U_*}{Wo} \cdot e^{\frac{Wo^2}{U_*^2}}$$

Where q_s : Quantity of suspended load per unit width and unit time

q : Discharge per unit width and unit time

Co : Concentration of river bed (ppm)

$\Delta F(w_o)$: Rate at which sand with settling velocity of Wo contains in the bottom material. (%)

c) Laursen's formula

$$q_T/U_*d = \frac{766}{265\psi_c}\left[\frac{\tau_c}{(\sigma/\rho - 1)gd} - \psi_c\right] \cdot f\left(\frac{U_*}{Wo}\right)$$

Where τ_c : Critical tractive force

ψ_c : Non-dimensional expression of critical tractive force

Wo : Settling velocity

d : Grain diameter

q_T : Total quantity of sediment per unit width and unit time (Bed material load + suspended load)

iii) Calculating conditions

The calculation of five flows (Q = 120, 400, 520, 900, and 1,100m^3/sec) is carried out. The constants necessary for calculation are as follows:

Kinematic viscosity	:	$\nu = 0.01$
Froud number	:	$f = 0.8$
Velocity head	:	$\alpha = 1.1$
Density of water	:	$\rho = 1.0$
Coefficient of roughness	:	$n = 0.033$(NO.0^k - No.16^k)
		$n = 0.036$(No.7^k - No.27^k)

iv) Calculation results

The calculation results are put in order and shown in Fig.-IV-32 through Fig.-IV-35.

Fig.-IV-33 which is the calculation results of bed-load, shows that the longitudinal change of bed-load is nearly equal to the changes of Froude number (Fr) and flow velocity (V). The sections are roughly divided into the section of 0^k to 5^k affected by the downstream back water, the mid-stream river course area from the neighborhood of 5^k up to 16^k and the sand arresting area in the upper stream above 16^k.

Local changes tend to occur at the fuses shifting from the National Road Bridge of No. 2.2^k and the sand arresting area in the neighborhood of 16^k to the river course. (Tendency of washing-away in the former and sedimentation in the latter).

On the other hand, the calculation results of suspended-load (Fig.-IV-34) show that it is extremely large on the downstream from the neighborhood of 5^k but on the upper stream from 5^k, it is nearly equal to the longitudinal change of bed-load. The calculation results of 5^k downstream are contradictory to the fact (If the calculation results are correct, the section of 5^k downstream should show a tendency of extreme washing-away. As reported previously, it is certain that the sedimentation in the neighborhood of the section should be a wash load which is evidently carried from the upper stream.
In addition, the logitudinal change of total runoff sediment volume which is calculated by the Rollsen's formula for the purpose of reference, shows a similar tendency except the case of 5^k downstream area, indicating the agreement of order.

As a result of the aforementioned calculation, it is concluded as follows:

1. The overall stability of deisgned river courses is secured more than expected initially.

2. Especially, with regard to the downstream river course section in the neighborhood of 16^k, the runoff sediment volume is continued in the section between 5^k to 16^k and it is found to be nearly stable except the local change in the neighborhood of 9^k, 11^k and 16^k (It is predicted that there exists some sedimentation in the neighborhood

of 9^k and 11^k, washing-away on the fuse downstream of 16^k and sedimentation on the fuse upper stream owing to the flash-board).

3. The section of 5^k downstream which is affected by back water of the River Gua-Gua shows a tendency of Sedimentation. Furthermore, it is assumed that as far as the judgement from the river bed sand, nearly all of the sedimentation is caused by wash load which is carried from the mountain side on the upper stream.

4. Concerning the sand arresting area on the upper stream from 16^k, according to the judgement from the results, the designed function (The sand is arrested at a stretch from the flow training stretch into the sand arresting basin.) is qualitatively satisfied because the sand arresting basin in the neighborhood of 20^k shows a tendency of sedimentation and the runoff sediment volume is increased at the flow training stretch from the valleys on the upper stream, although there remains a problem of one-dimensional dealing with the unequal flow and run-off sediment formula.

5. Degree of effects of both bed-load and suspended load on change of the river bed is same (on the 6^k upper stream, approximately in order of 10^{-1} with the bed-load and also with the suspended load respectively).

FIGURE III-35-1 DIAMETER OF 0% PARTICLE

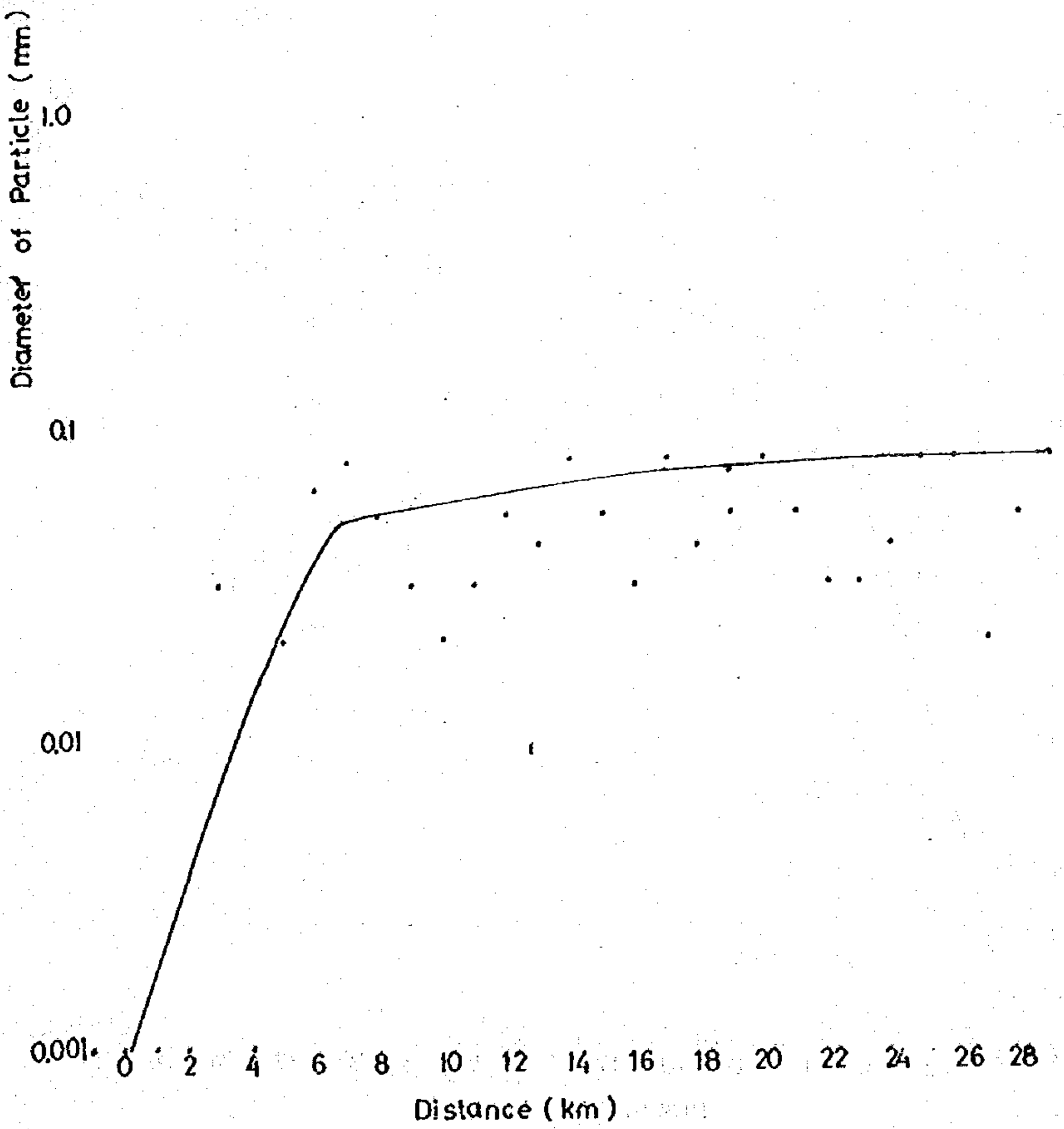


FIGURE III-35-2 DIAMETER OF 25% PARTICLE

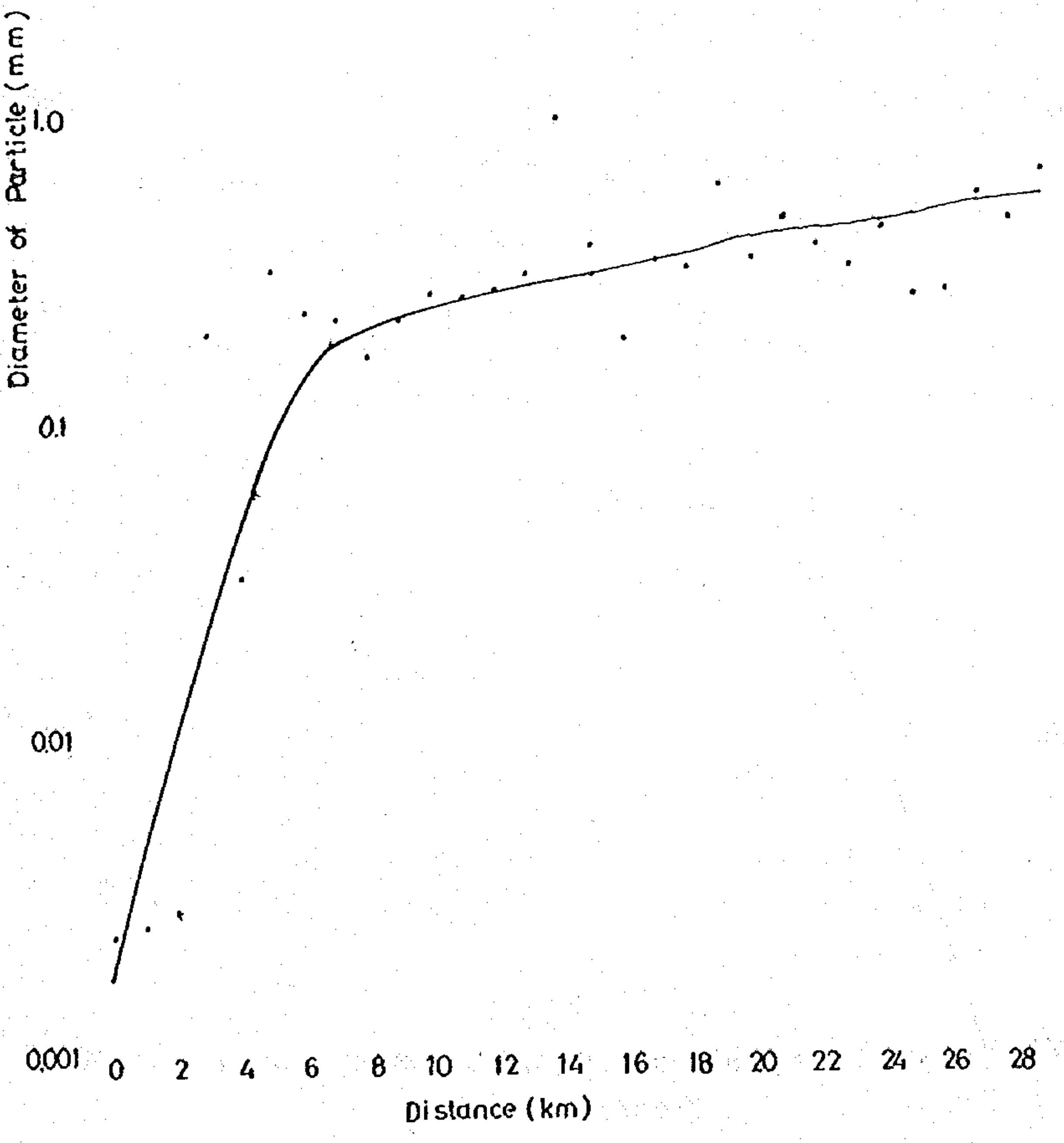


FIGURE Ⅲ-35-3 DIAMETER OF 50% PARTICLE

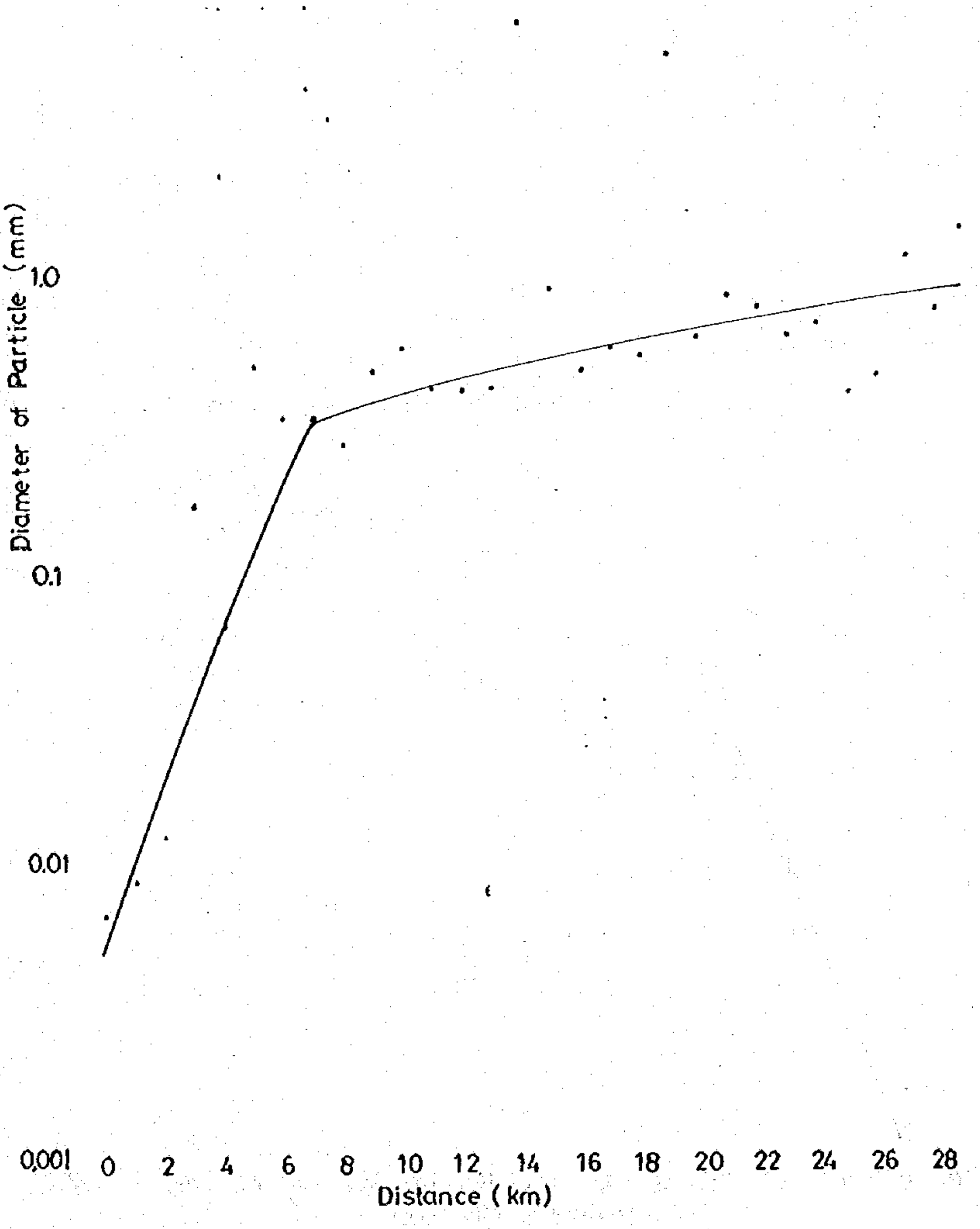


FIGURE III-35-4 DIAMETER OF 75% PARTICLE

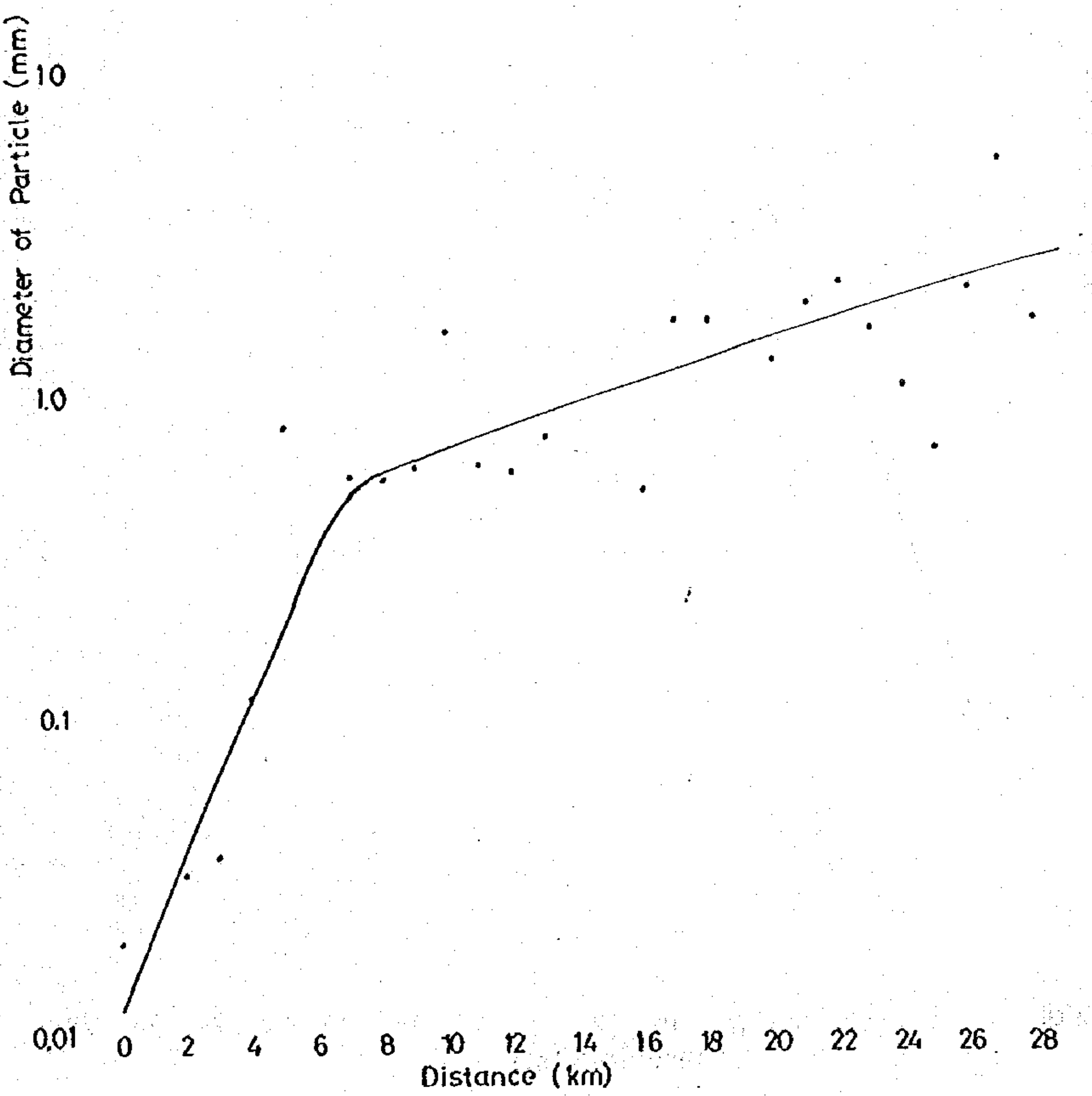


FIGURE Ⅲ-35-5 DIAMETER OF 100% PARTICLE

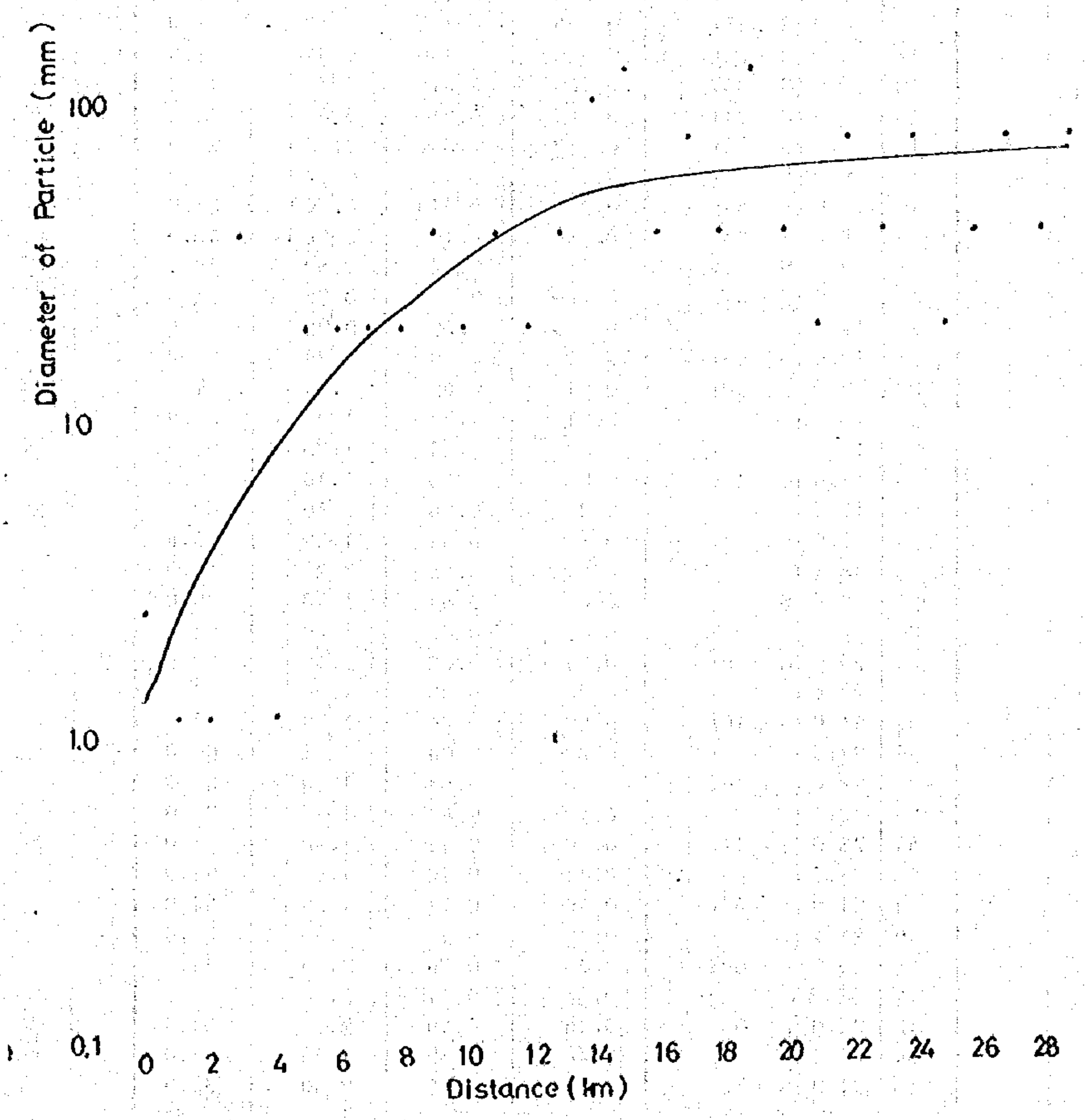


TABLE III-17 GRADING DISTRIBUTION

NO.	STATION NO	D 25%	D 50%	D 75%	D 100%
1	0.0 (S - 27)	0.0020	0.0054	0.012	1.3
2	0.7 ( )	0.0032	0.0084	0.018	1.9
3	1.0 (S - 29)	0.0040	0 0100	0.020	2.4
4	1.8 ( )	0.0090	0.016	0.032	3.4
5	2.0 (S - 25)	0.0100	0.019	0.036	3.6
6	2.3 ( )	0.0135	0.023	0.043	4.2
7	3.0 (S - 32)	0.023	0.038	0.064	5.5
8	4.0 (S - 33)	0.049	0.068	0.110	8.0
9	4.2 ( )	0.055	0.074	0.120	8.4
10	5.0 (S - 48)	0.090	0.115	0.190	11.0
11	6.0 (S - 39)	0.140	0.200	0.330	14.0
12	7.0 (S - 35)	0.180	0.310	0.480	18.0
13	8.0 (S - 31)	0.200	0.340	0.540	22.0
14	9.0 (S - 36)	0.210	0.360	0.590	26.0
15	10.0 (S - 21)	0.230	0.380	0.650	31.0
16	11.0 (S - 64)	0.240	0.410	0.700	36.0
17	12.0 (S - 65)	0.260	0.44	0.76	42.0
18	13.0 (S - 46)	0.270	0.46	0.82	48.0
19	14.0 (S - 57)	0.280	0.48	0.88	52.0
20	14.8 ( )	0.290	0.50	0.92	54.0
21	15.0 (S - 63)	0.295	0.50	0.94	54.0
22	16.0 (S - 30)	0.31	0.52	1.00	57.0
23	16.2 ( )	0.31	0.53	1.05	58.0
24	16.4 ( )	0.32	0.54	1.06	58.0
25	17.0 (S - 47)	0.32	0.55	1.10	59.0
26	17.4 ( )	0.32	0.56	1.15	60.0
27	18.0 (S - 11)	0.34	0.57	1.20	60.0
28	18.4 ( )	0.34	0.58	1.25	60.0
29	19.0 (S - 19)	0.35	0.59	1.30	60.0
30	19.4 ( )	0.36	0.60	1.40	61.0
31	20.0 (S - 9)	0.37	0.62	1.45	62.0
32	20.4 ( )	0.38	0.63	1.50	62.0
33	21.0 (S - 10)	0.39	0.64	1.60	63.0
34	21.4 ( )	0.39	0.66	1.64	64.0
35	22.0 (S - 16)	0.40	0.68	1.70	64.0
36	22.4 ( )	0.40	0.69	1.75	64.0
37	23.0 (S - 44)	0.40	0.71	1.80	64.0
38	23.4 ( )	0.41	0.72	1.85	64.0
39	24.0 (S - 54)	0.42	0.74	1.90	64.0
40	24.4 ( )	0.42	0.74	2.00	64.0
41	25.0 (S - 60)	0.43	0.75	2.05	64.0
42	25.4 ( )	0.44	0.77	2.10	64.0
43	26.0 (S - 55)	0.45	0.79	2.20	64.0
44	26.4 ( )	0.46	0.80	2.25	65.0
45	27.0 (S - 12)	0.47	0.82	2.30	65.0

FIGURE Ⅲ-36 FROUD NUMBEB, VARIATION WIDE, VELOCITY PLANED VIVER CONISE

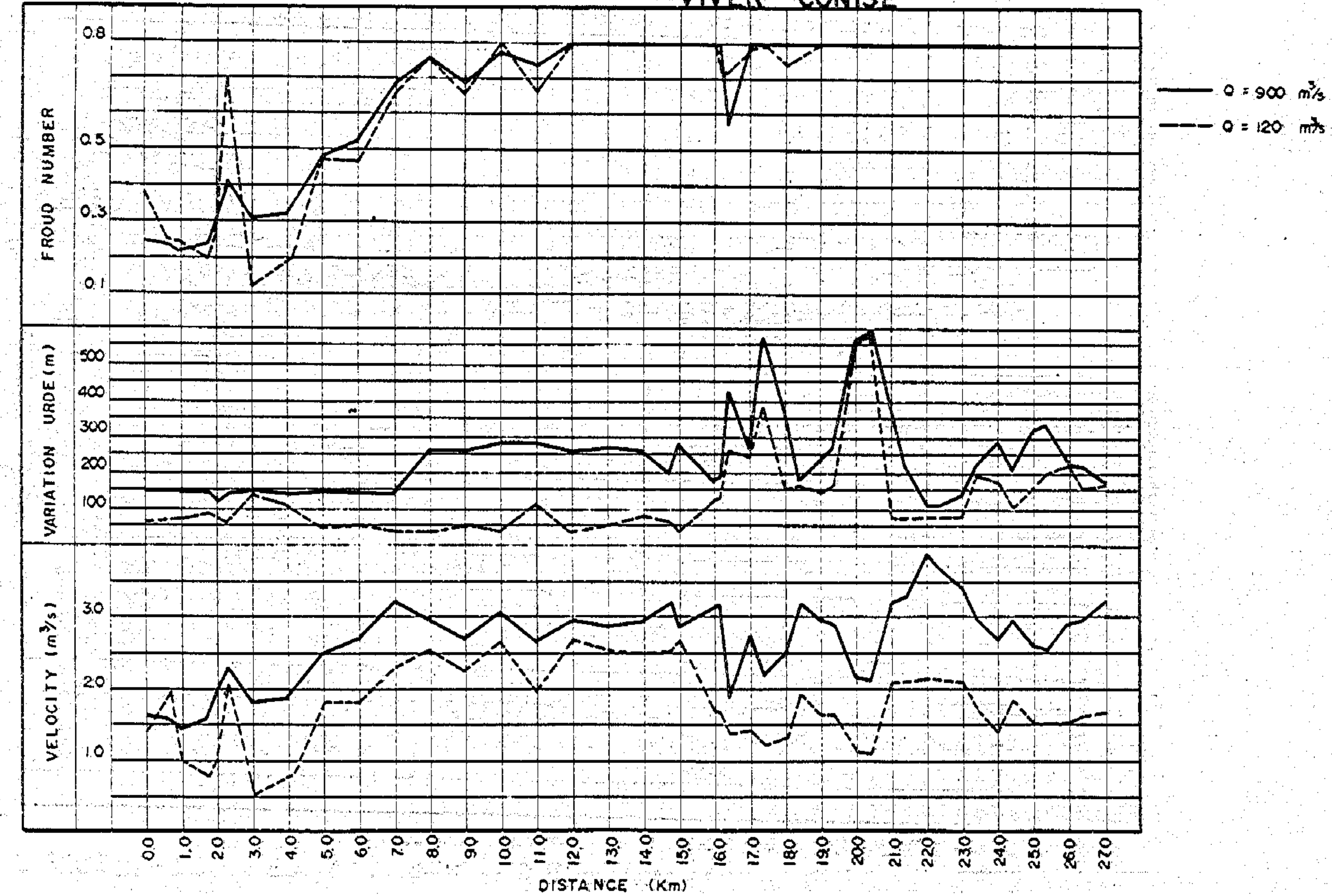


FIGURE III-37 BED MATERIAL LOAD
— SATO, KIKUKAWA AND ASHIDA FORMULA —

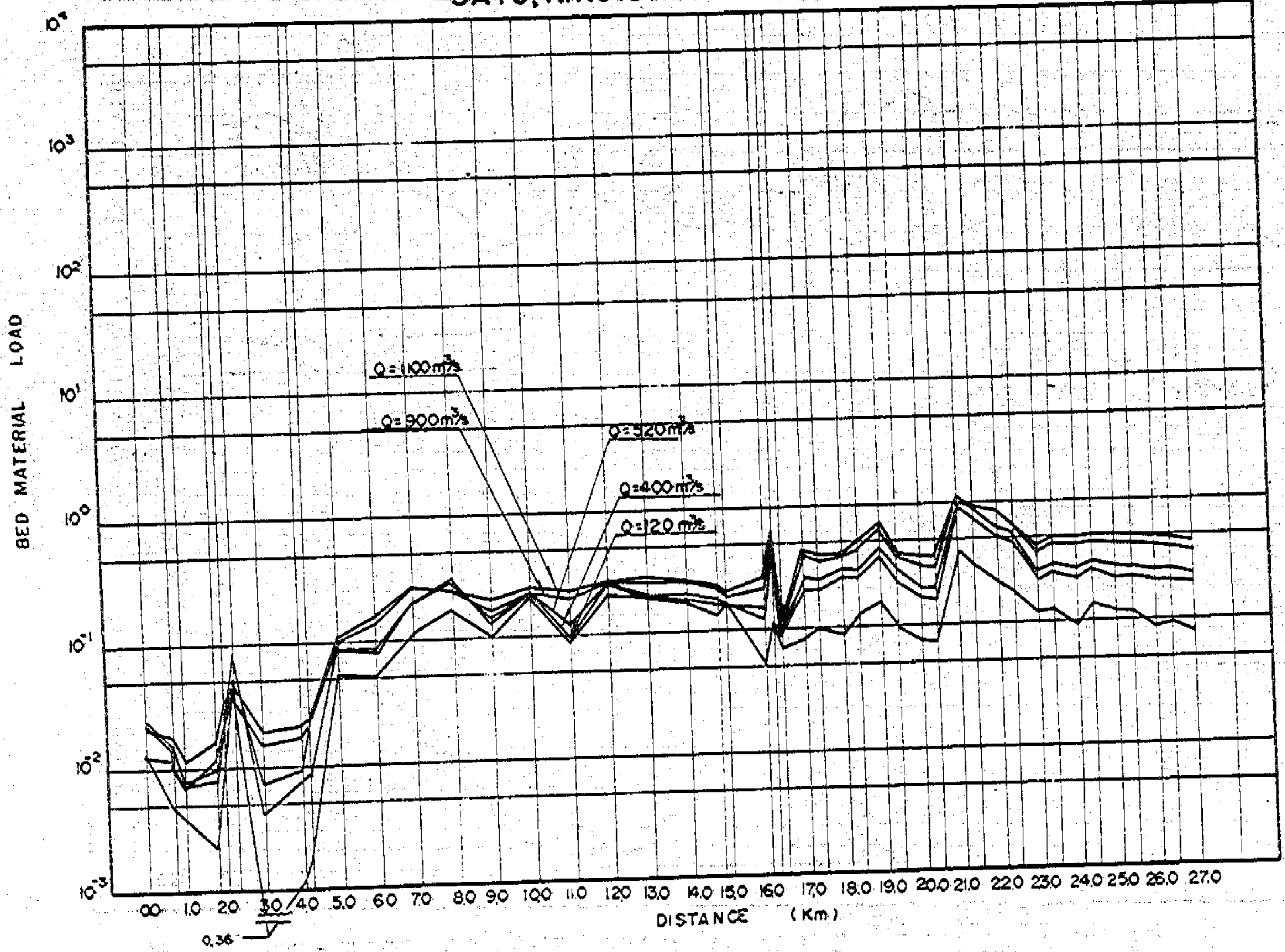


FIGURE Ⅲ-38 SUSPENDED LOAD —LANE - KALINSKE FORMULA—

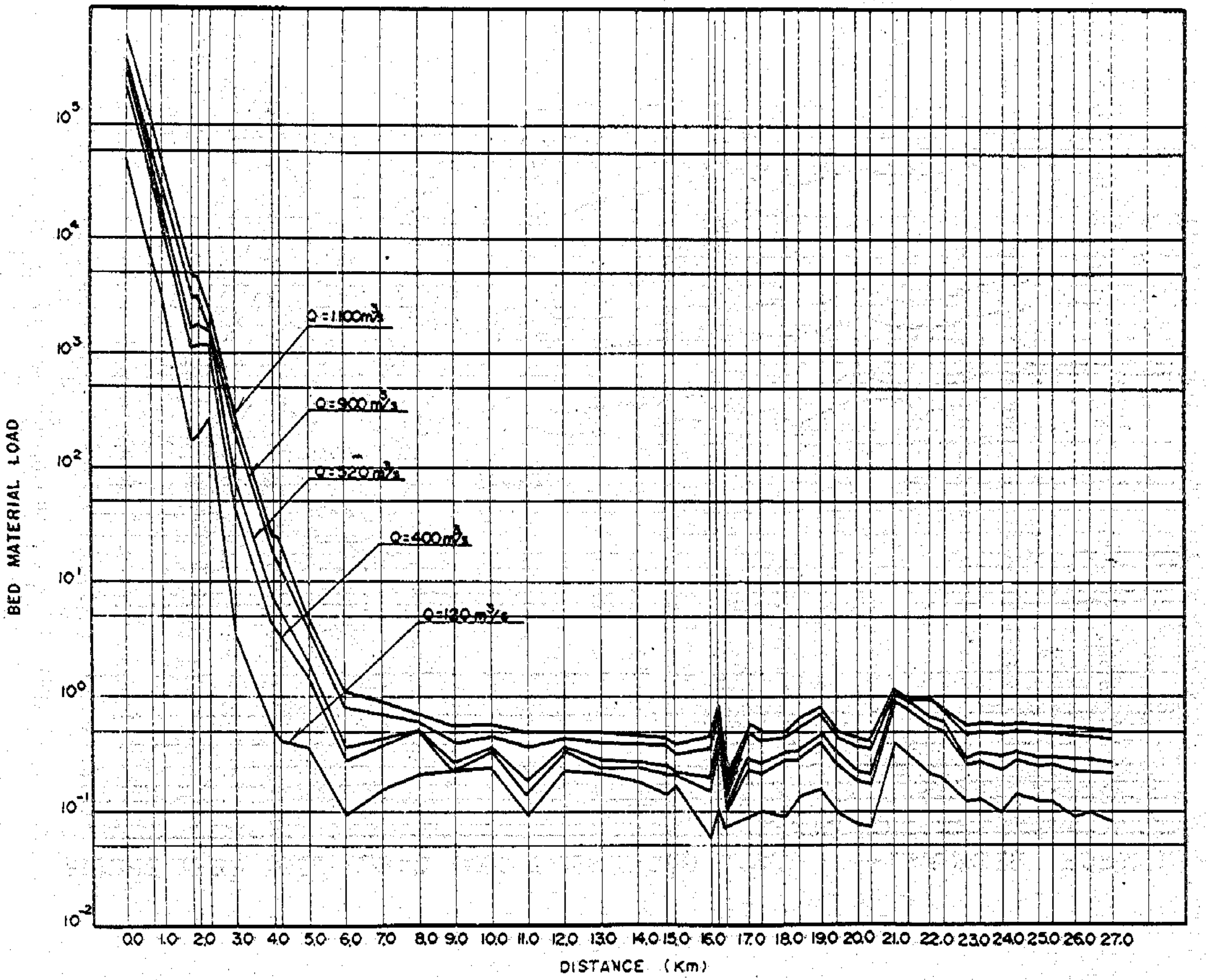


FIGURE Ⅲ-39 BED LOAD AND SUSPENDED LOAD —LAURSEN FORMULA—