so we should either use circular piles (as a pier) with spacing of more than 3 b or we use the formula which accounts for all possible river approaches to the pier, such as

$$ds/b = 1.43\ (do/b)^{.3} \times f_1.$$

where f_1 is a factor for the shape of the pier. It is prudent to use this equation to be on the safer side.

4.8 Relationships with Reynold No. or Froude No.

Chitale's formula (1941) based on the results of extension of the original Poona Model tests of the Hardings bridge on GANGES river:

$$d/sb = 6.65\ F - .51 - 5.49\ F^2$$

where F is the Froude No.

Shen Formula (1969) See Fig. 23.5

$$ds = .00073.\ Re^{.619}\ \text{(upper envelope of data)}.$$

where Re = Reynold number for the pier = $\bar{u}b/\nu$

4.9 Feeling of Confidence in any prediction Method

Because, so few field measurements of scour have been made and those few leave something to be desired, the various methods of predicting scour cannot be checked against reality and so to build a feeling of confidence in any predictive method or even to compare different methods of prediction of scour field measurements is a must.

The programmes of field measurement at sites of simple geometry, at sites of complex geometry, measurements in large river, during moderatly high flow and measurements during floods are needed. The programmes are also handicaped because many bridges are protected with stone and it is not possible to measure scour in the rare floods.

4.10 Observations of Prototype Scour in Indian Rivers

(a) Observations in the period 1924-1942 (Inglis 1949).

Total scoured depth was measured from the water surface to the bottom of scour and so includes scour due to contraction and local scour due to the piers. The depths were compared with Lacey Regime Depth.

$$d_{Lacey} = .473 \ (Q/f)^{1/3}$$

where $f = 1.76 \sqrt{\text{median grain size in mm}}$.

The individual values of the ratio of actual scour/d Lacey varied between 1.73 and 2.62. The average value was 2.09 with r.m.s. values of 12.9%.For design purpose the inglis Lacey equation is

$$do+ds = .95 \ (Q/f)^{1/3}.$$

For estimation of scour 'f' is left out of the equation and in the case, the individual ratios vary from 1.72 to 2.59. With average value of 2.0 with r.m.s. of 8.45%.

In these observations the grain size varied from .17 to .37 mm (f .71 to 1.10) and discharge varied from 850 to 63000 m^3/s.

(b) Observations in the 1967 Report:

After careful considerations 8 bridges out of 48 originally planned were selected. The bridges had spans from 9-23 metres,bed dia varied from 1 to 3 mm and discharges varied from 35-600 m^3/s. The computed Lacey depths varied from 1.35 to 3 metres and total scour varied from 2.3 to 5.5 metres. For currents upto 35^o angle of attack the maximum scour occurred on the side of the pier under attack and was equal to $1.99 x d_{Lacey}$ on the average (r m s value of 15%).

The combined data gave $\dfrac{do + ds}{\text{Lacey depth}} = 1.93$

(c) 1968 Report - Ganga Pul at Makameh

The bridge has 14 spans of 123 metre, the width of the wells is given as 9.75 m. The observation for the period 1958 - 1967 were 18 for Monsoon discharges 5000-34000 m^3/s. With corresponding Lacey depth = 7.5 to 14.5 m using f = 1.15.

Average scour depth at the nose was equal to 1.75 x d_{Lacey} against side scour of 2.15 d_{Lacey}.

The interesting observation was that the scour in non-Monsoon was more due to lower silt contents. The scour at nose for 13 observations was 2.54 d_{Lacey} against corresponding Side scour of 2.7d_{Lacey} but these scours were however lesser than the largest in Monsoon floods.

(d) 1972 Report

The report includes 50 more observations on 4 out of 8 bridges mentioned in 1967 report. For discharges varying from 60-500 m^3/s. With d_{Lacey} of 1.5 to 3 metres and total scour of 2.1 to 6.8 metres. In 36 out of 50 observations the scour depth was measured and correlated with Lacey depth.

The observed values of scour below water levels are plotted against the calculated values of Lacey's Regime Depth with

$$d_{Lacey} = .47\ (Q/f)^{1/3} \text{ (metre units) Fig.24.1}$$
$$d_{Lacey} = 1.34\ (q^2/f)^{1/3} \text{ (metre units) Fig.24.2}$$

The extreme values of scour are given by relations

$$\frac{do + ds}{.47\ (Q/f)^{1/3}} = 2.55$$

$$\frac{do + ds}{1.34\,(q^2/f)^{1/3}} = 2.14$$

The other correlations for average values of scour are:

$$\frac{do + ds}{.47(Q/f)^{1/3}} = 1.92 \quad \text{r.m.s. value of } 15\%$$

$$\frac{do + ds}{1.34(q^2/f)^{1/3}} = 1.46 \quad \text{r.m.s. value of } 15\%$$

For design purpose the maximum values should be used as some unknowns are involved.

4.11 Field Measurements on S Kunk River by Laurson & Toch 1956

The prototype and the model data are compared in Fig. 25. The scale model gave a good correspondence with prototype.

4.12 Scour Protection around the Pier

The fundamentals of this protection are:

(a) To break the incident current and in this way weaken the vortex generating the erosion.

(b) To contain the horse shoe vortex inside an enclosure.

(c) To redirect the diving flow and to lift it up.

(d) The flow velocity giving the initial scour at the base of pier is equal to half that corresponding to general bed load movement or in other words the size of stone for protection of scour should be for the velocity double than the extrence flood velocity in the river in the approaches to the bridge. The diameter of stone can be worked from the Izbash(1935) Formula:

$$2\ V\ max = 5\sqrt{Ds} \quad \text{(m.units)}.$$

where D_s = dia of stone

To achieve the objective Chabert and Engeldigner investigated a circular pier founded on a circular Caisson and they concluded that if the dia of caisson is three times the diameter of the pier and top of Caisson is about half the diameter of pier below the natural bed the resultant scour around the pier reduces to only one third the value without the caisson.

Shen and Schneider (1970) tried a caisson surrounded by a vertical lip (in an attempt to reduce the dimensions of the caisson). TANAKA 1969 and Thomas 1967 proved that the reduction of scour depth upto 50% can be obtained by placing a horizontal flat plate with a diameter of atleast 3 times the pile diameter placed .3 - .4b below the undisturbed bed level.

Riprap of the desired dia and weight can replace the caisson if the width of the stone apron around the pier is increased to twice the pier width to permit launching of the stone apron at its outer periphery. The thickness of stone is suggested to be atleast 3 times the theoretical dia from the equation:

$$2\,V_{max} = 5\sqrt{Ds} \quad \text{m. units.}$$

The top of riprap should be below the normal bed level to avoid excessive exposure and should be preferably about half of pier thickness below the normal undisturbed bed.

4.13 Typical Example of New Road Bridge Over Jhelum

The design discharge	= 900,000 cfs.
The design features are given in Fig.26.	
Design H.F.L.	= 756
Waterway	= 22 Bays each 147' span.
R.L.of transm slab	= 740.0

The retrogression upto 10' is expected due to Mangla reservoir so it is assumed that the effective width of piers is at wells and is 25' (dia of wells) and for the same reason the effective waterway = 20x122+145 = 2585 ft. as there is every likelihood of remaining choked of side bays by the guide bank and its apron.

The effective average discharge per foot run = 348.1 cusecs

The value of Depth of scour from water surface $= K_1 \, q^{2/3}$

The value of K_1 read out from Fig. 11 for channel No.3 of the basic study (which is reasonable assumption in view of control point higher up) is 1.3.

Depth of scour from water surface = 64.3.

4.14 The scour depth due to contraction by Laursen method

It is known that the bankful discharge in Jhelum river after the construction of Mangla Dam is of the order of 200,000 cusecs.

The depth of flow for 200,000 cusecs

$$= .47 \, (Q/f)^{1/3} = 27.49$$

The basic test of the model of river Jhelum constructed for new road bridge had shown that as the discharge increases from 200,000 to 900,000 the depth of water increases by 6.5 and so that depth of water in main channel at 900,000 cusecs will be = 27.49 + 6.5

= 33.99

Say = 34 ft.

The discharge in the main channel for a depth of 34 ft. works out to be 378400 cusecs by Lacey formula assuming f = 1.

From Laurson formula (as the guide banks are not encroaching the main channel).

$$\frac{\text{Depth from water surface to scour R.L.}}{34} = \left(\frac{900000}{378400}\right)^{6/7} = 1.899$$

The depth of scour from water surface =65.55

Therefore the depth of scour below bed level =64.55-34 = 30.55

The corresponding depth of scour below bed level by Dr.Mushtaq Ahmad's Formula=64.3 - 34.0 = 30.30.

The values of scour from the two methods are in agreement.

The localized scour due to pier effect is given by the equation:

$$ds/b = 1.43\ (do/b)^{.3}$$

$$ds/25 = 1.43\ (34/25)^{.3}$$

$$ds = 39.20$$

The total scour from water surface works out to be = 64.3+39.20 = 103.5 ft.

This scour is 2.44 times the Lacey depth. Soil exploration at site had shown that at a depth of about 40-50 ft. the samples contained upto 40% of Cobbles and boulders upto 8" and so ultimately the bed of scour will be paved with Cobbles and the ultimate scour will be less.

The H.F.L. at the Railway bridge is at elevation 756 and the Railway record shows that the scour has not gone below elevation 720, and thus the maximum scour is 36.0' and assuming Lacey Inglis Formula the value of R = 18.0. At the Railway bridge the intensity of flow for 900,000 cusecs is 185 cusecs and so the value of 'f' from Lacey relationship works out to be 4.277.

The contraction depth of scour from LAURSEN Formula works out to be 36.6 ft. and the value of local scour now works out to be 33.65 and thus the total scour works out to be 70.23 ft.

The bottom elevation of well is 671 and so from H.F.L. of 756 the grip depth = 756-671-70.23 = 14.77 Say = 15.0

5.0 CONCLUSIONS

1. It is neither economically feasible nor technically advisable to bridge the entire width of the stream upto the flood limits. The confining of river flow-way unduly may cause excessive back-water with resultant flood hazards, create excessive scour endangering the foundations and may involve excessive funds for maintaining the approach embankments to the bridge.

Laboratory investigations on the specific studies and the basic study on the optimum water-way of a bridge on fluvial channels have indicated that the minimum water-way of a bridge is about 1.2 times the Lacey's width for the design discharge of the bridge located on an alluvial channel.

2. The guide banks projecting upstream from the abutments re-direct the flow pattern to parallel the desired channel course, to even out the flow at the bridge, to protect the road embankments within river Khadir against parallel flow and to minimise the scour at the piers.

The effectiveness of the guide banks is a function of the geometry and shape of guide banks, the quantum of flow on the flood plain, the river approach to the guide banks, the size of bridge opening, the placement of the approach embankments and the lay out of marginal embankments limiting flood plains.

The guide banks if not suitably aligned for the worst river approach gets captured at one of its upstream noses and when fully captured a deep embayment forms behind the captured nose which sweeps off the main current away from the captured guide bank towards the opposite guide bank where it develops a deep channel. Thus the spans along the captured guide bank get masked by a sandy bela whereas the spans along the opposite guide bank are overtaxed.

The Laboratory investigations on specific studies and basic study have indicated that:

a) the minimum length of the guide banks projected on the axis of the bridge is three-fourth to full width of the bridge(L).

b) The U/S noses of the two guide banks should be so apart that the embayment behind the captured guide bank should not sweep off the current on to the opposite bank but preferably in the area between the axis of the bridge and the captured guide bank and this objective is achieved if the noses are atleast 2L apart.

c) The alignment of the guide banks in between its U/S (right or left) nose and the abutment (right or left) may be parabola or elliptical or a compound curve of radii equal to 2.5L and .5L at divergence angle of 14° to 15° from the abutment line.

3. In rivers of wide Khadir the possibility of the river going behind the guide bank into the still water area at its back and eroding the approach embankment cannot be ruled out thereby entailing river training works higher up. The requisite geometry of spur heads is discussed by the Authors in their paper No. 428 of Engineering Congress (Proceedings of 1975).

4. The scour that may occur at a bridge waterway can be categorised as follows:-

a) The scour that would occur in the fluvial channel as a result of degradation of channel bed with or without a bridge crossing.

b) The scour that may occur generally at the bridge waterway because the flow is contracted by the bridge crossing.

c) The local scour that occurs because of the distortion of the flow pattern in the immediate vicinity of the bridge piers and abutments.

In the absence of evidence to the contrary, the scour of these different categories must be treated as additive.

5. The local scour at the bridge pier may be due to diving flow at the pier, or horse shoe vortax system or wake vortax system and is a function of pier width, shape of pier, concentration of flow and angle of attack. The maximum local scour that occurs because of the distortion of the flow pattern in the immediate vicinity of the bridge piers is given by the equation:

$$ds/b = 1.43 \quad (do/b)^{.3}$$

where ds is the scour below the bed of approach channel

do = depth of the approach flow
b = width of the pier

The above relationship is the equation of the upper enveloping curve for the data given by the different Investigators.

An average curve of the data is given by:

$$ds/b = 1.5 \tanh do/b.$$

6. The over bank flood plain flow returns to the main channel at the guide bank heads thereby augmenting the discharge of the channel and consequently the capability of the flow to carry more sediment. The magnitude of the scour at the bridge water way as a consequent of encroachment of flood plain and/or main channel by the approach embankments can be assessed by the Laursen method or discharge intensity formula method as described in paragraph 2.2.1 and 2.2.2

7. The observation of the prototype scour in alluvial rivers has indicated that:-

The extreme value of scour are given by relations:

$$\frac{do + ds}{.47(Q/f)^{1/3}} = 2.55$$

$$\frac{do + ds}{1.34(q^2/f)^{1/3}} = 2.14$$

8. The Laboratory experiments have indicated that the localised scour can be reduced upto 50% by providing a solid or stone apron around the pier placed .3b to .4b below the undisturbed bed level in a width equal to the width of the pier.

TABLE-I

TABLE SHOWING WATERWAY OF VARIOUS BRIDGES ON RIVERS OF PAKISTAN

Rivers	Name of the Bridge	Q max. design	Lacey waterway $P=2.67\sqrt{Q}$	Clear waterway (W).	Looseness Factor $(K)=\frac{W}{P}$	Safe Scour level	H.F.L record
RAVI.	Jassar	275000	1400	1751	1.25	739.00	792.5
	Shahdara (old)	260000	1335	1350	1.01	650.00	695.1
	Shahdara (new)	350000	1620	1537	0.95	-	695.0
	Chichawatni	200000	1195	1054	0.90	-	516.42
	Mari Pattan	230000	1280	1270.75	0.99	-	577.00
	Abdul Hakim	150000	1034	455.0	0.44	427.00	470.1
CHENAB	Alexandra	1000000	2230	2240	1.01	694.00	754.0
	Talibwala	935000	2600	2996	1.15		628.50
	Chiniot (east)	350000	1580	568.8	0.36	488.8	598.00
	Chiniot (west)	450000	1791	765.5	0.42	514.30	597.50
	Rivaz	730000	2281	2200	0.97	466.00	522.70
	Shershah	600000	2236	3400	1.47	344.00	391.75
JHELUM	Jhelum (old)	941000	2590	4518	1.71	719.85	759.00
	Jhelum (new)	900000	2540	3002	1.18	-	756.00
	Victoria	809800	2403	2550	1.06	624.00	677.63
	Khushab	600000	2068	2160	1.04		600.50
SUTLEJ	Adamwan	400600	1690	1778	1.05	315.00	381.00
INDUS	Attock	633000	2124	1209	1.00	Gorge	66.50
	Khushal Garh	900000	2540	728	0.28	Gorge	802.32
	Kalabagh	1200000	2925	2462.33	0.84	655.00	670.00
	Sukkur	800000	2393	1399	0.58	Gorge	204.90
	Kotri	1000000	2670	1755.50	0.65	Gorge	71.00
HUB	Road Bridge	325000	1522	1500	0.98		80.00

TABLE -II

TABLE SHOWING DISCHARGE PASSING THROUGH EACH BAY OF THE NEW BRIDGE AT 500,000 CS.INCOMING RIVER DISCHARGE

No. of bays	Discharge in cusecs through each bay		
	24 bay bridge (Cusecs	23 bay bridge (Cusecs)	22 bay bridge (Cusecs)
1.	8,000	-	
2.	18,000	18,000	-
3.	12,000	10,000	8,000
4.	14,000	12,000	10,000
5.	14,000	14,000	13,000
6.	15,000	14,000	14,000
7.	15,500	16,000	18,000
8.	16,000	19,00	21,000
9.	15,000	18,000	22,000
10.	18,000	18,000	22,000
11.	20,000	20,000	23,000
12.	20,000	23,000	25,000
13.	21,000	22,000	26,000
14.	22,000	22,000	26,000
15.	25,000	23,000	28,000
16.	25,500	27,000	28,000
17.	26,000	30,000	29,000
18.	25,000	29,000	29,000
19.	26,000	28,000	28,000
20.	26,000	30,000	30,000
21.	28,000	32,000	29,000
22.	30,000	30,000	25,000
23.	30,000	27,000	20,000
24.	20,000	18,000	16,000

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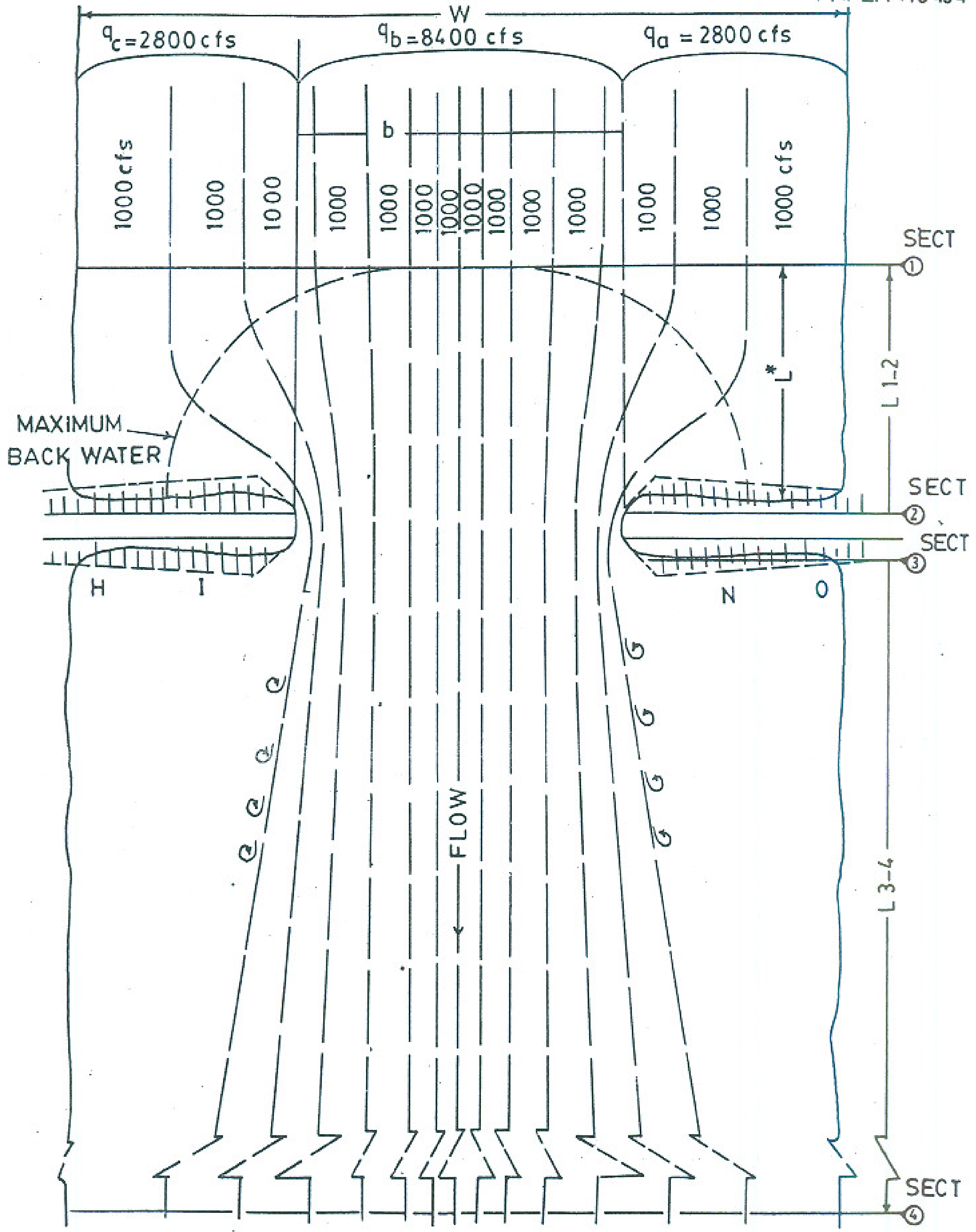
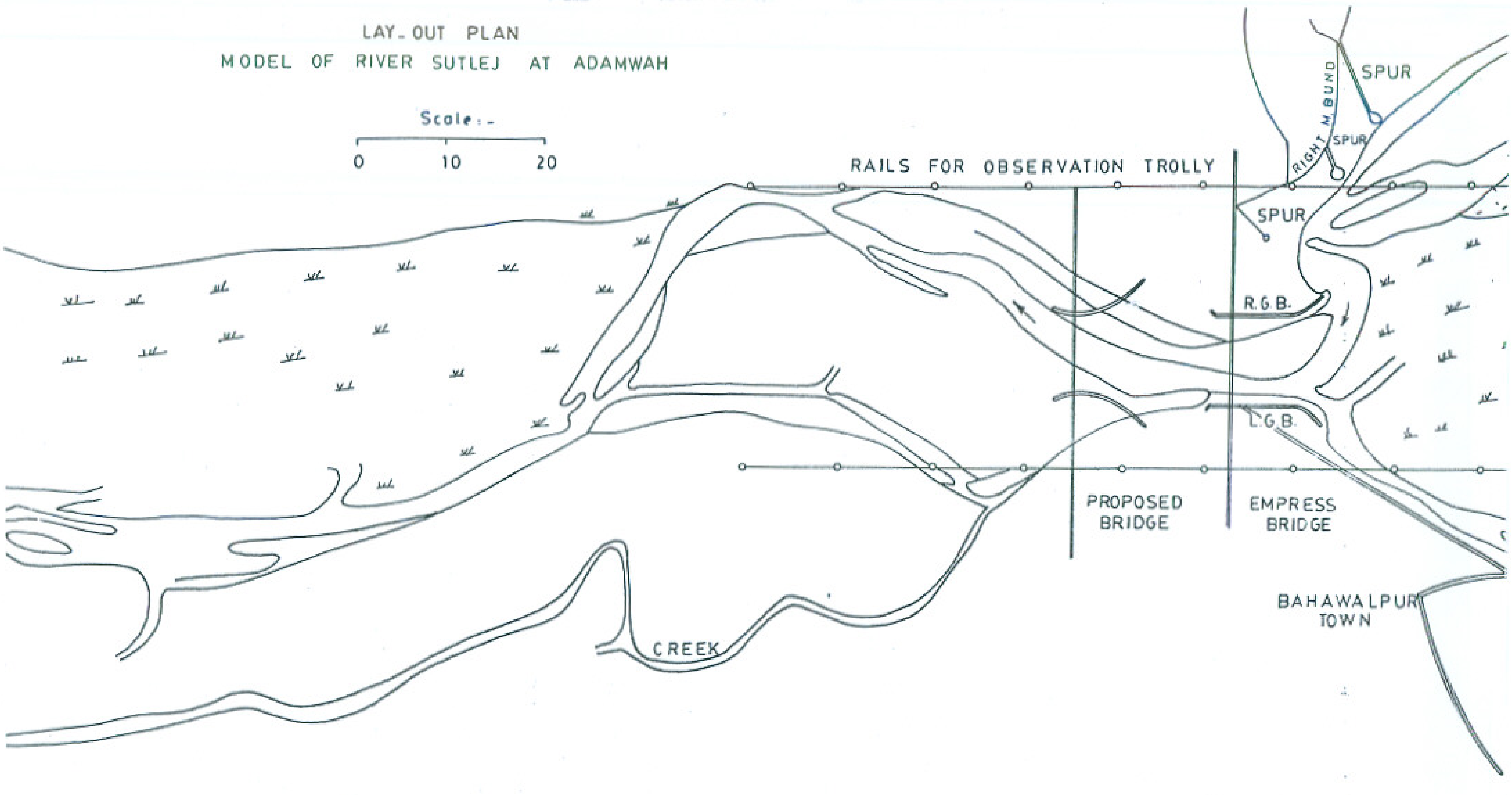
Fig. 1
PAPER NO 434



FLOW LINES FOR TYPICAL NORMAL CROSSING

LAY-OUT PLAN

MODEL OF RIVER SUTLEJ AT ADAMWAH



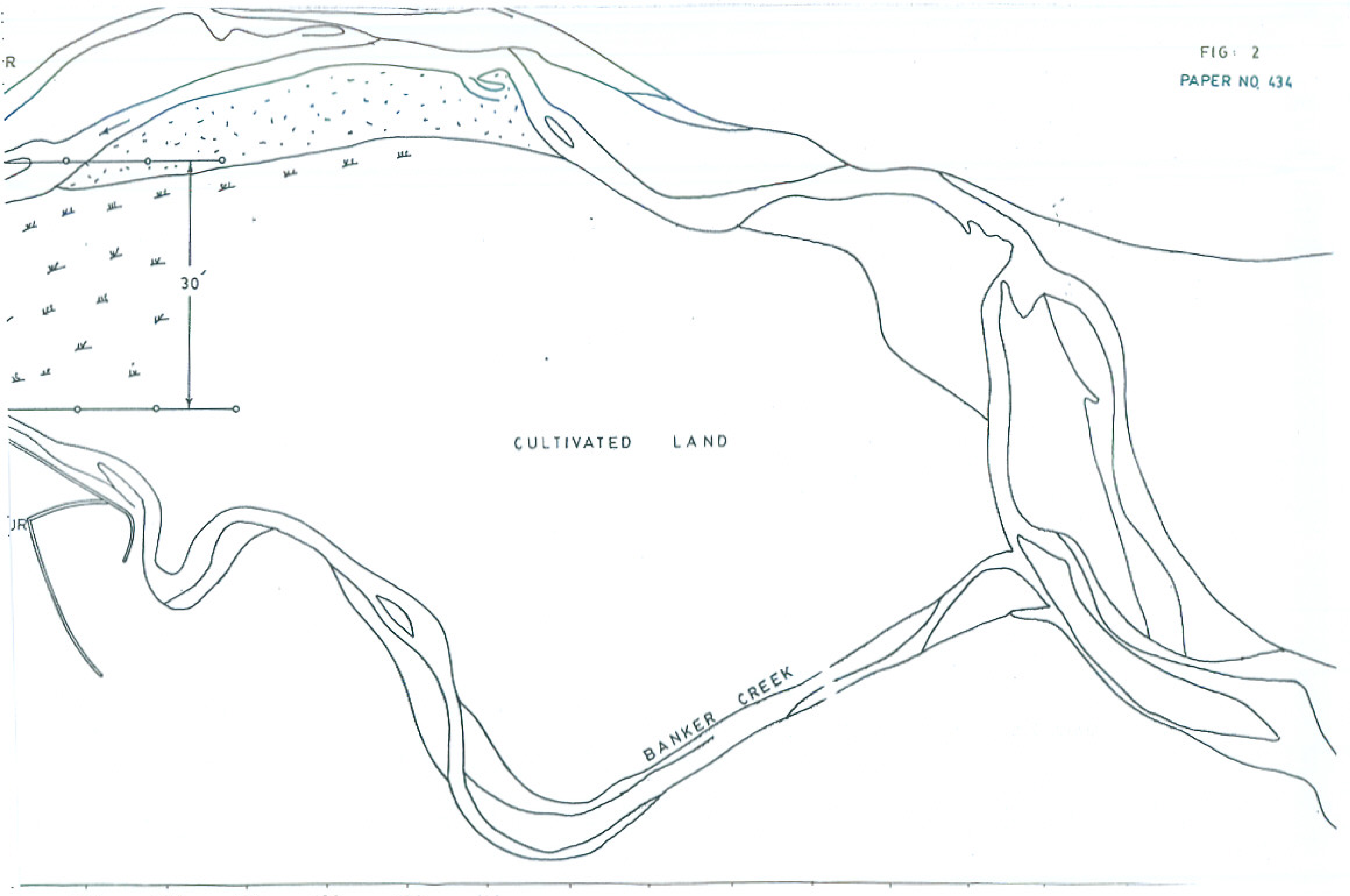


FIG: 2
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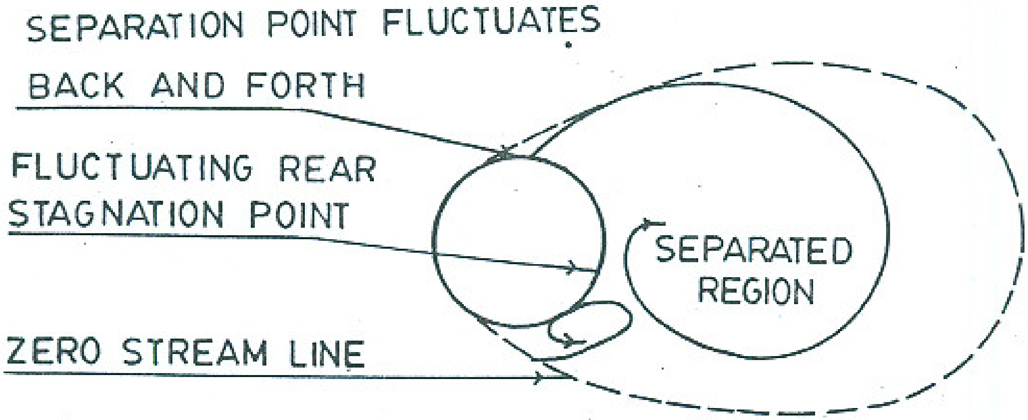
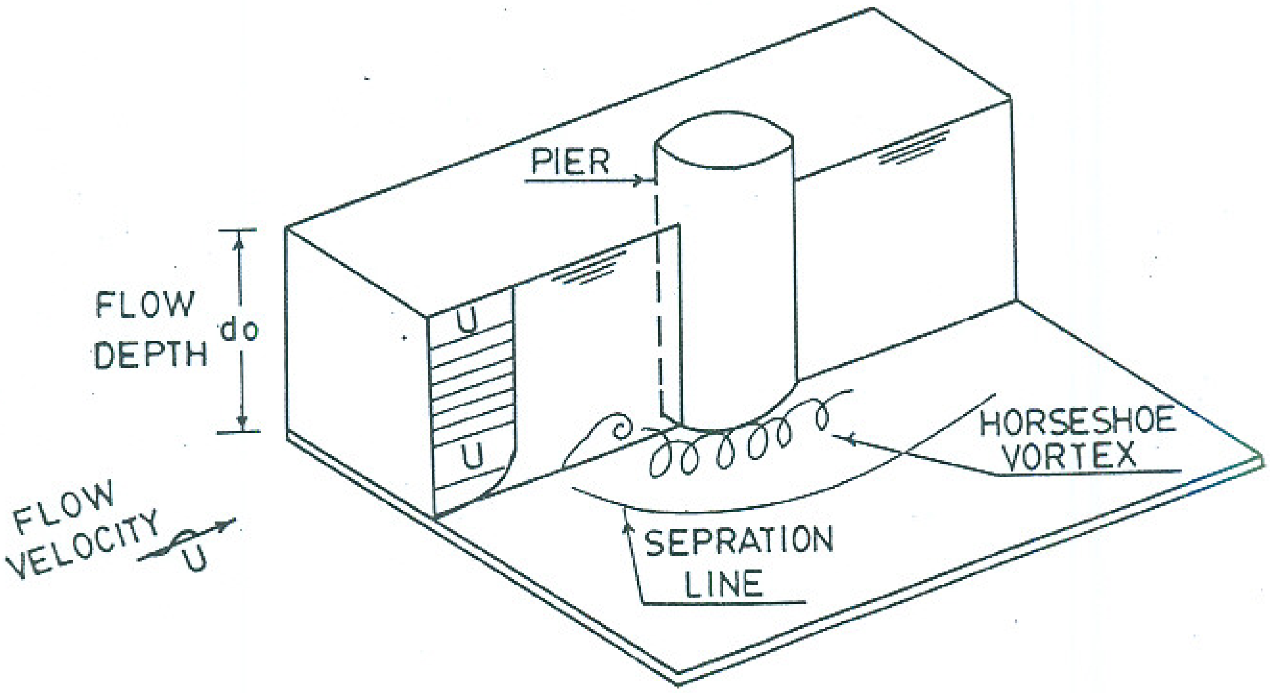
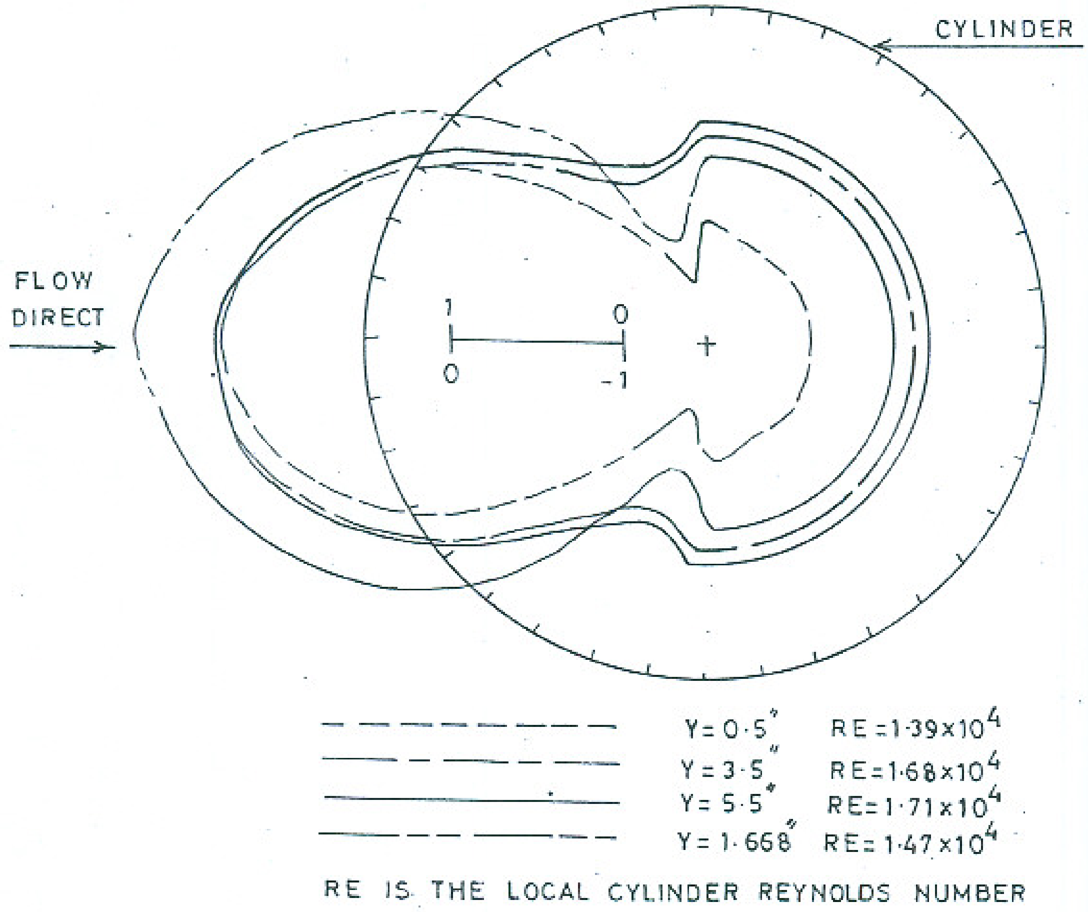


SEPARATION AND OSCILLATION BEHIND CYLINDER (TOP WIEW)
(AFTER PETRYK 1969)



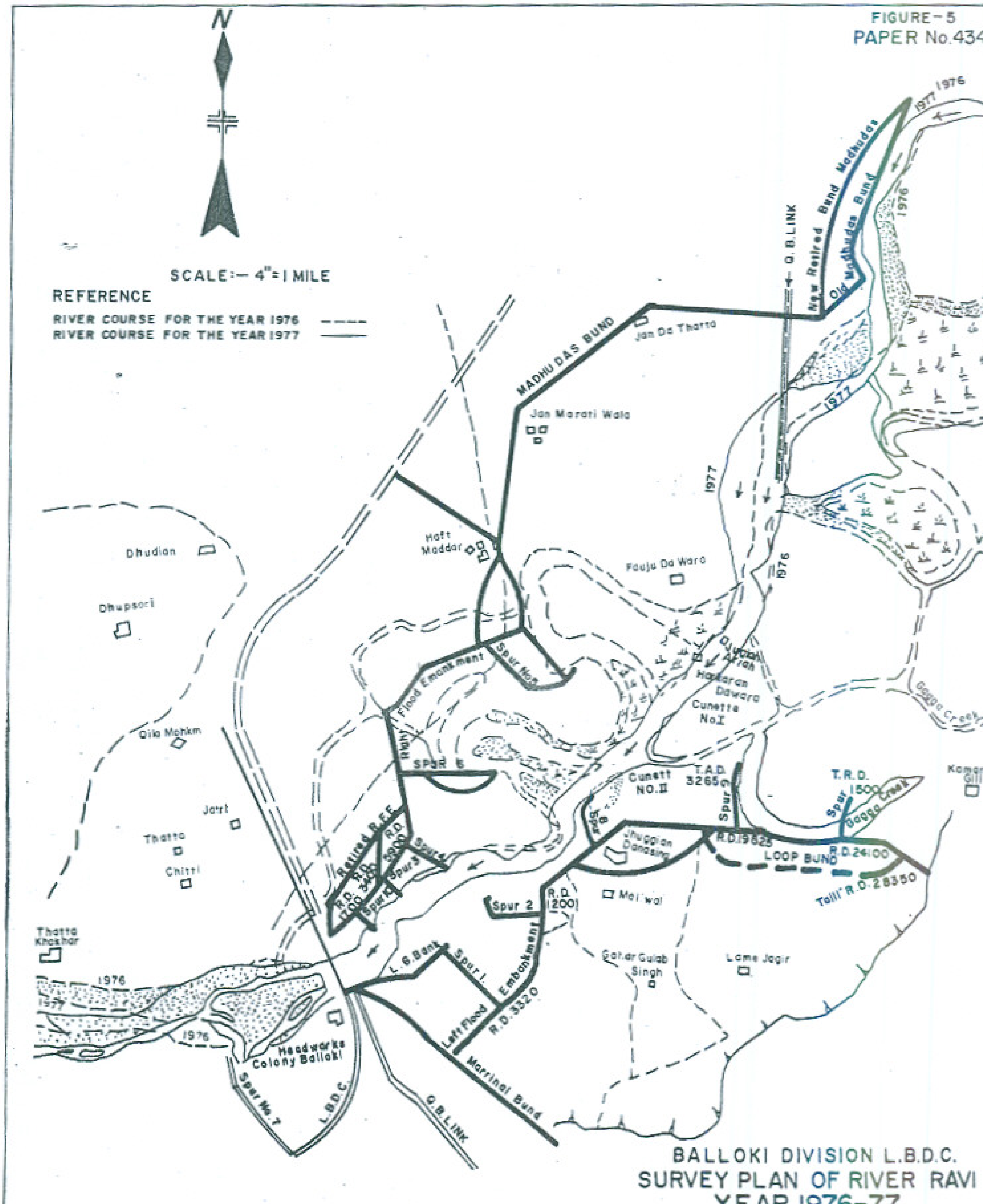
SKETCH OF HORSESHOE VORTEX

Fig 4
PAPER NO 434

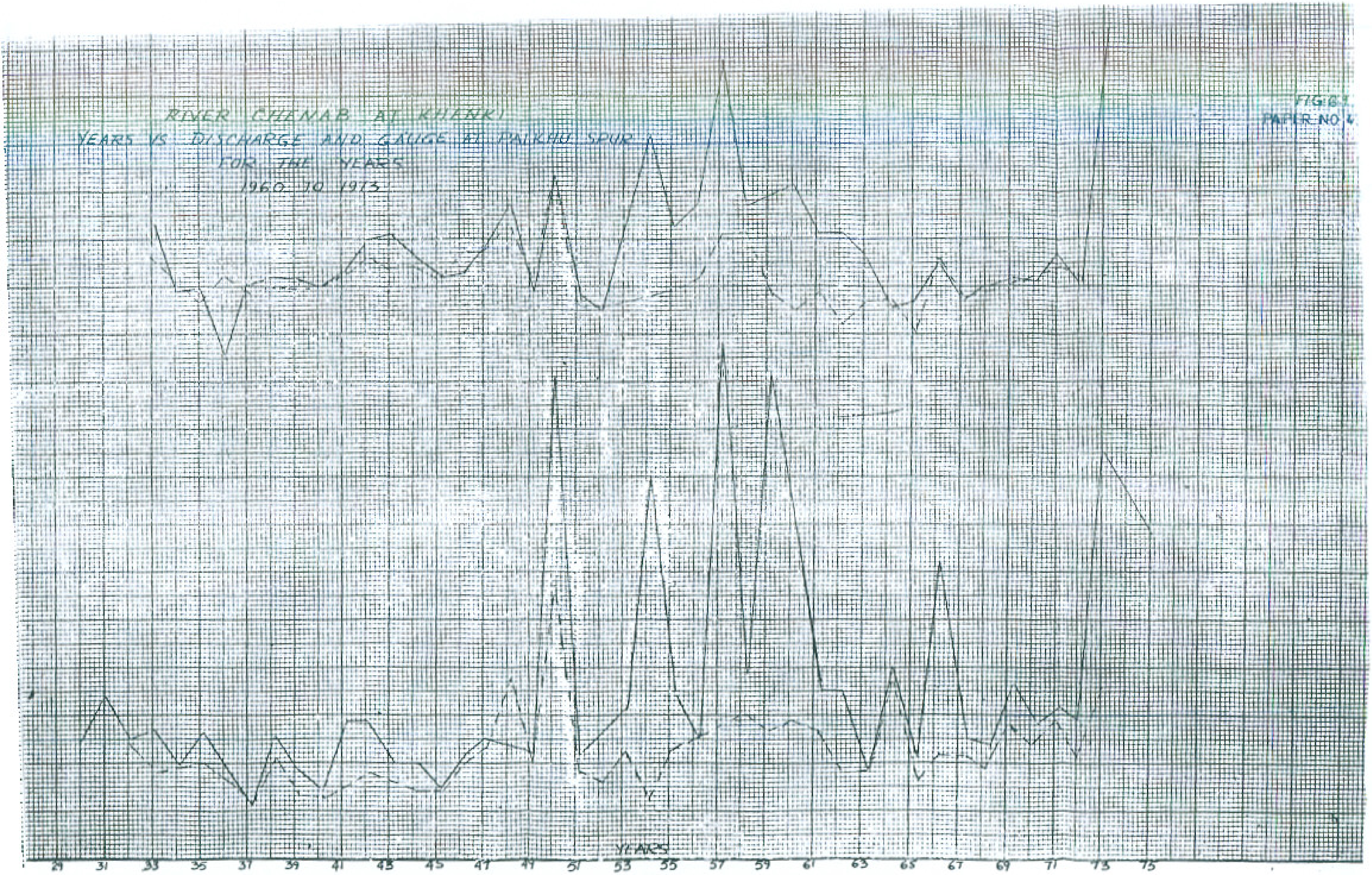


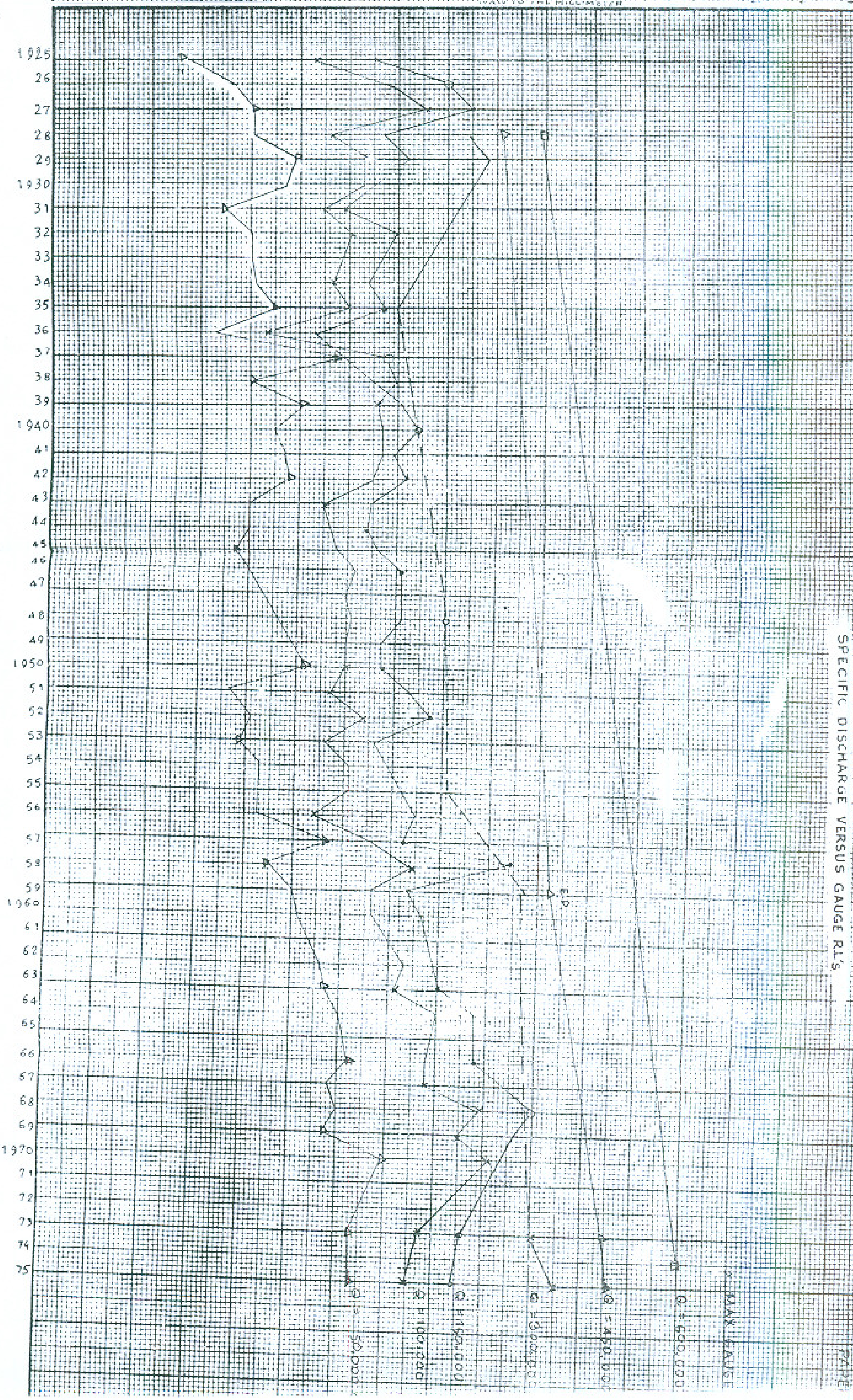
PRESSURE COEFFICIENT Cp

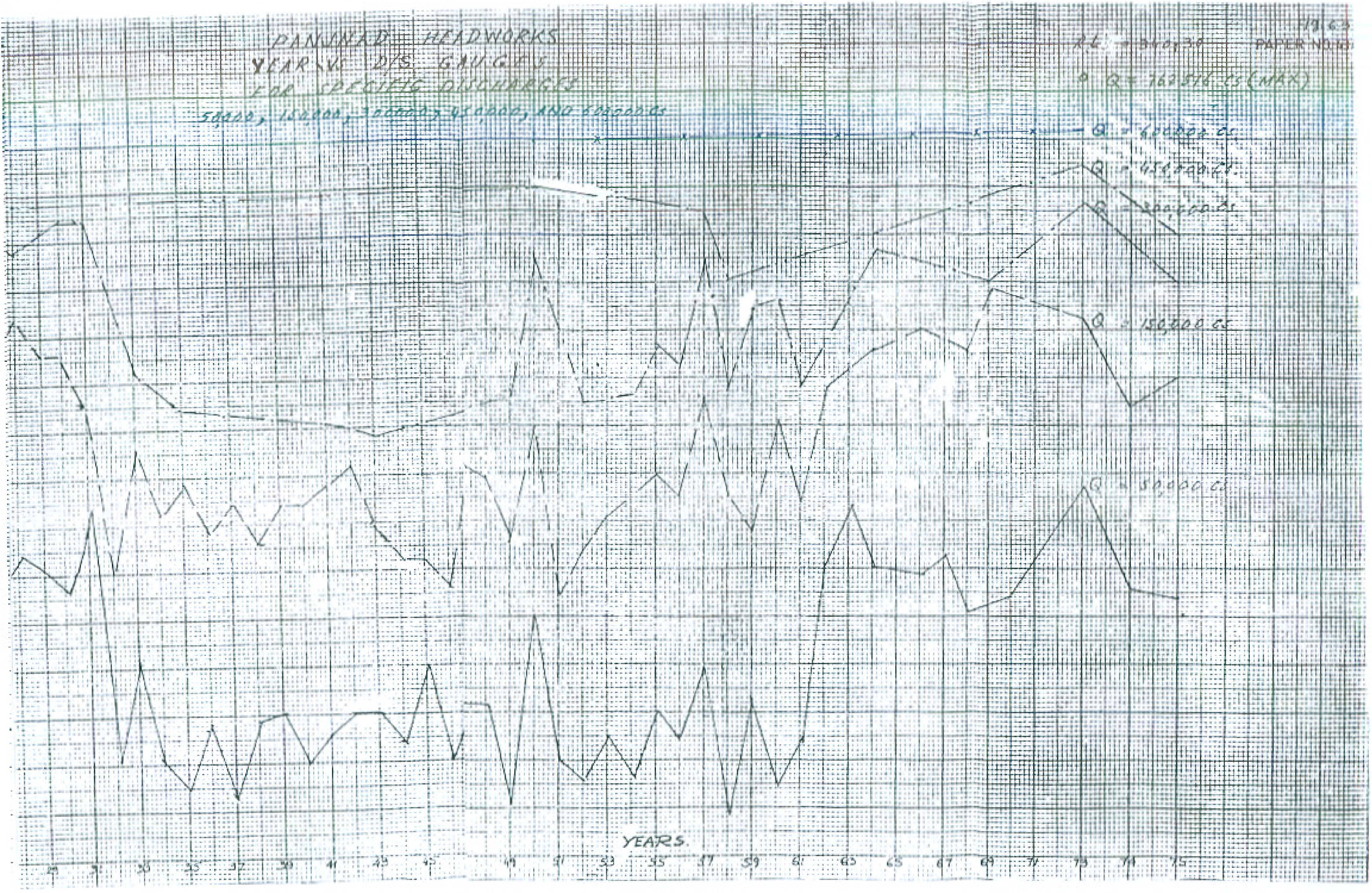
FIGURE-5
PAPER No.434

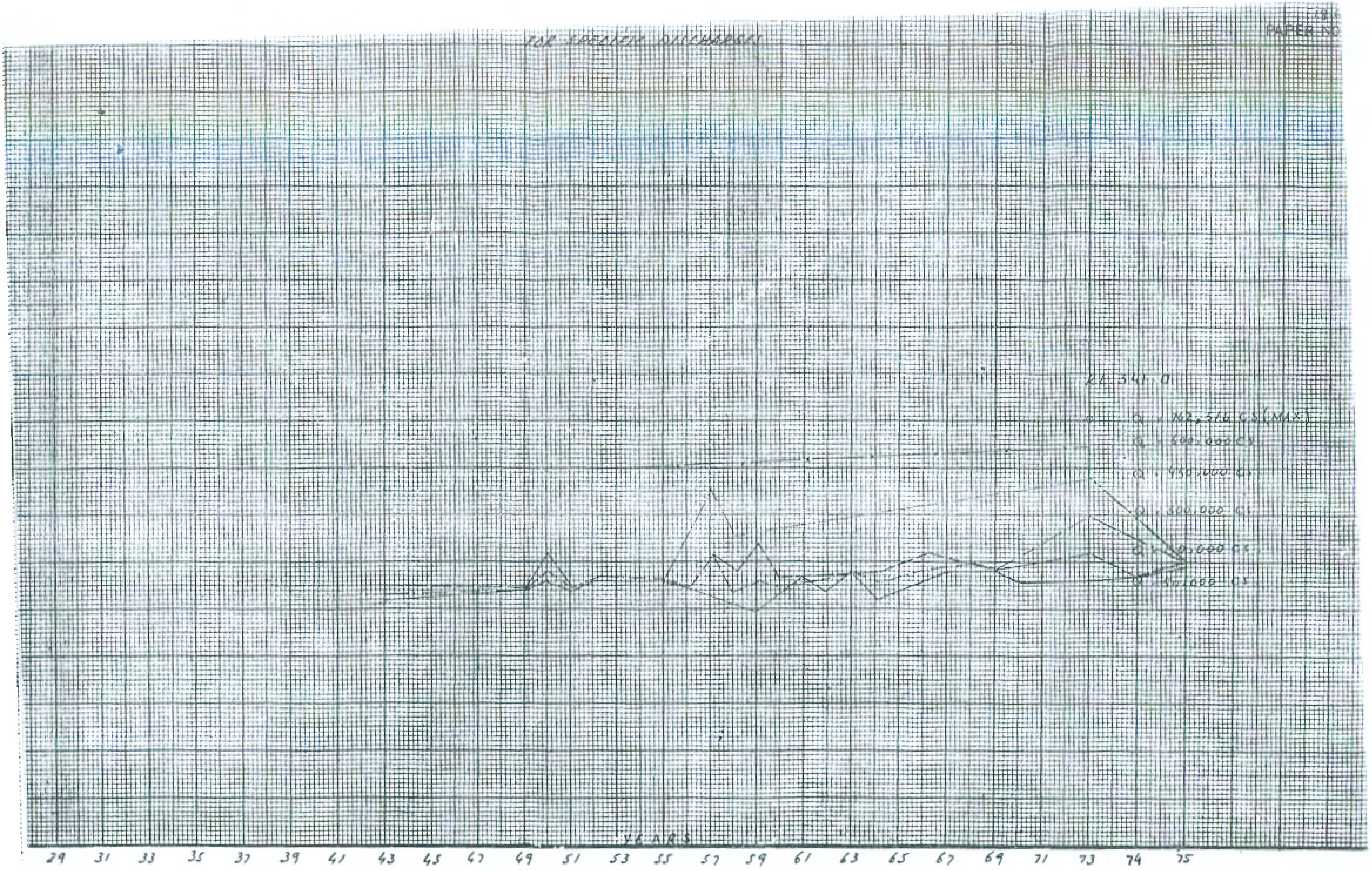


BALLOKI DIVISION L.B.D.C.
SURVEY PLAN OF RIVER RAVI
YEAR 1976-77


RIVER CHENAB AT KHANKI
YEARS VS DISCHARGE AND GAUGE AT PAIKHU SPUR
FOR THE YEARS
1960 TO 1973
FIG 6.4
PAPER NO 4
YEARS
29
31
33
35
37
39
41
43
45
47
49
51
53
55
57
59
61
63
65
67
69
71
73
75

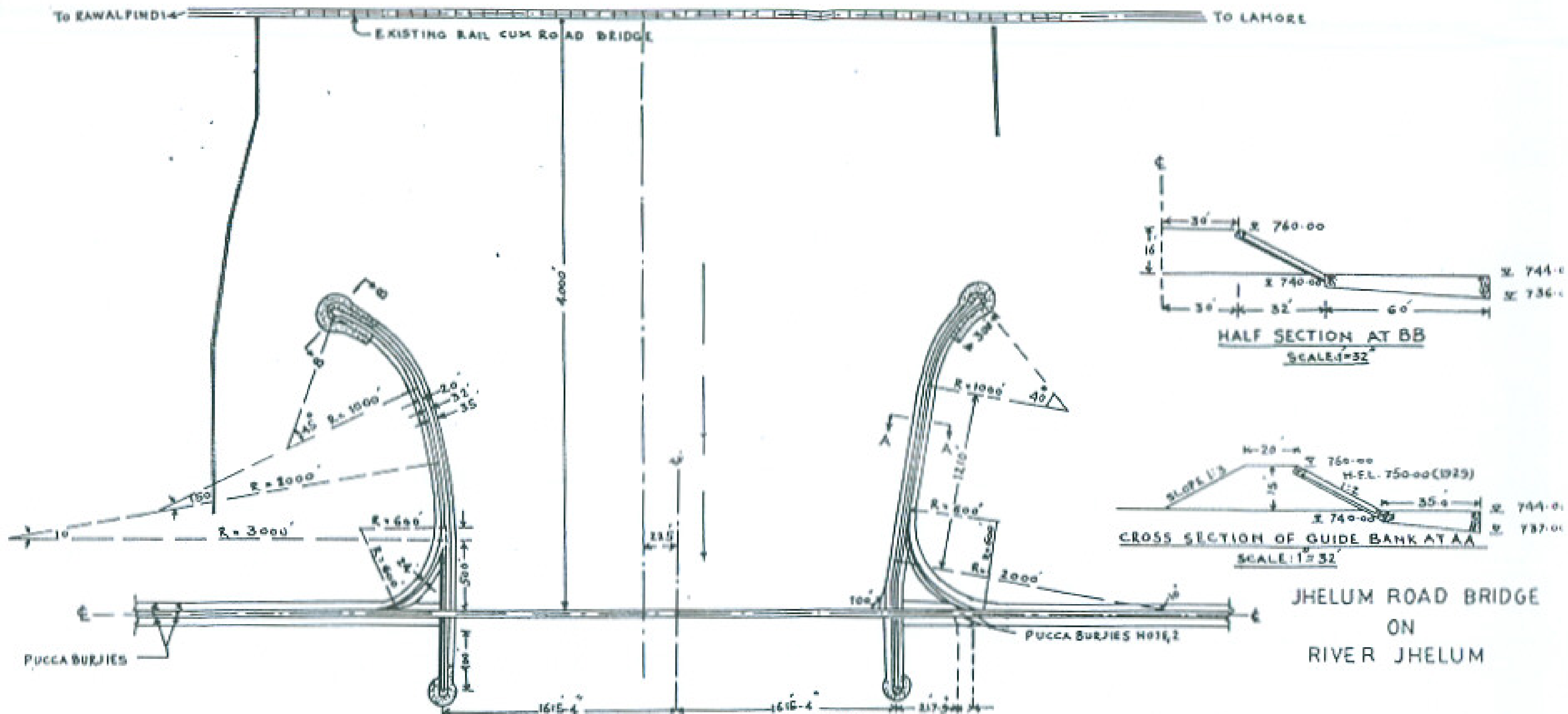

CHENAB AT RIVAZ RAILWAY BRIDGE
SPECIFIC DISCHARGE VERSUS GAUGE R.L's
1925
26
27
28
29
1930
31
32
33
34
35
36
37
38
39
1940
41
42
43
44
45
46
47
48
49
1950
51
52
53
54
55
56
57
58
59
1960
61
62
63
64
65
66
67
68
69
1970
71
72
73
74
75
Q = 50,000
Q = 100,000
Q = 150,000
Q = 300,000
Q = 400,000
Q = 600,000
MAX GAUGE


PANJNAD HEADWORKS
YEAR VS D/S GAUGES
FOR SPECIFIC DISCHARGES
50,000, 150,000, 300,000, 450,000, AND 600,000 cs
R.L. = 340.30
Q = 762,576 cs (MAX)
Q = 600,000 cs
Q = 450,000 cs
Q = 300,000 cs
Q = 150,000 cs
Q = 50,000 cs
YEARS
29
31
33
35
37
39
41
43
45
49
51
53
55
57
59
61
63
65
67
69
71
73
74
75



PAPER NO
Q : 462,576 CS (MAX)
Q : 400,000 CS
Q : 450,000 CS
Q : 300,000 CS
YEARS
29
31
33
35
37
39
41
43
45
47
49
51
53
55
57
59
61
63
65
67
69
71
73
74
75

FIG 7·1
PAPER NO 434



DETAILS OF LAY OUT & GUIDE BAINKS

JHELUM ROAD BRIDGE
ON
RIVER JHELUM

NEW ROAD BRIDGE ON RIVER JHELUM
FIGURE SHOWING WATER LEVELS UPSTREAM OF THE EXISTING BRIDGE WITH PROPOSED BRIDGE HAVING 24, 23, 22, 21 & 20 BAYS PLACED 4000′ BELOW THE RAILWAY BRIDGE

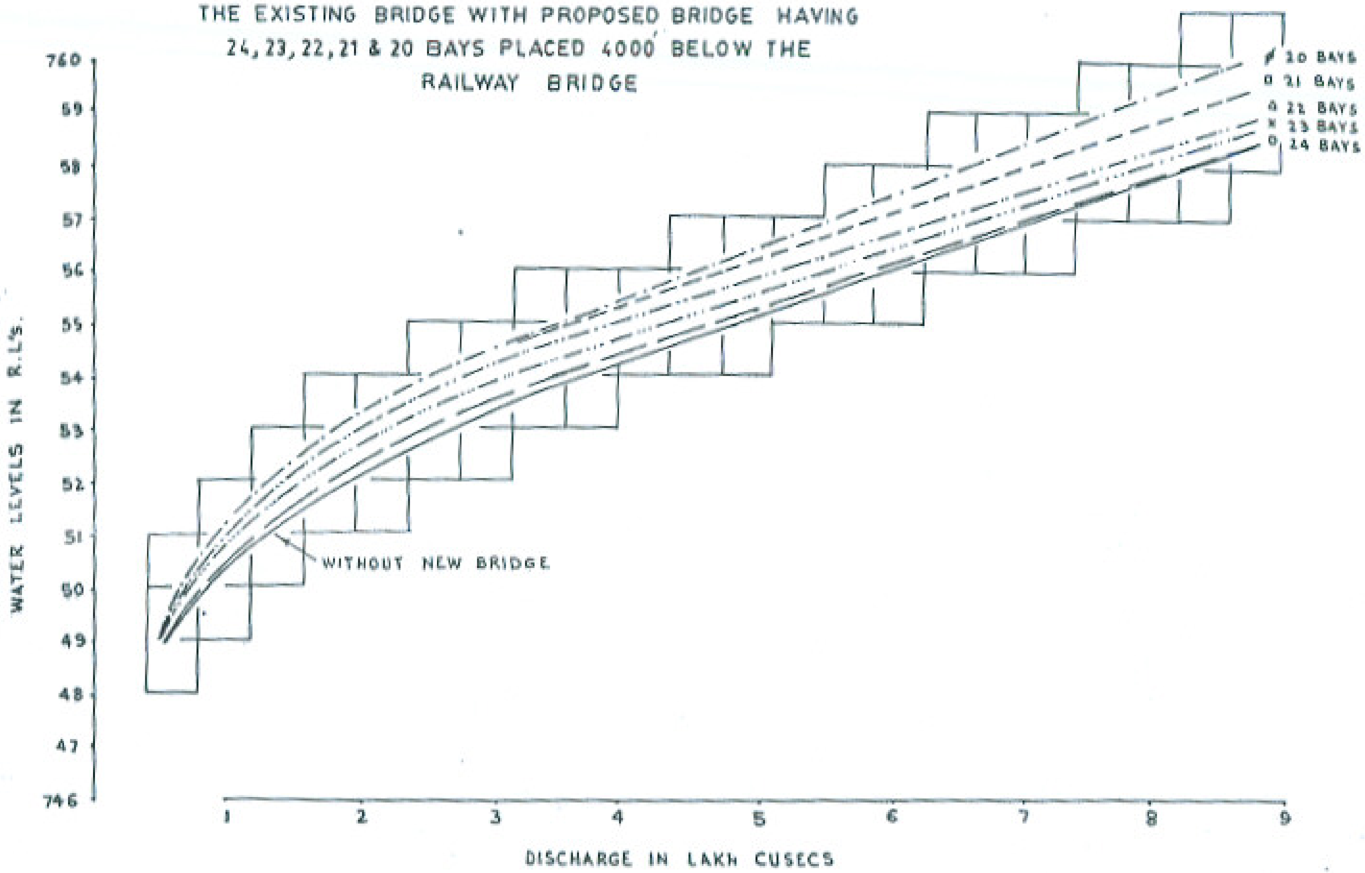


FIG. 8.1

PAPER No.

MODEL OF RIVER CHENAB AT ALEXANDRA BRIDGE
WATER LEVELS AGAINST ADDITIONAL WATERWAY
AT DIFFERENT DISCHARGES

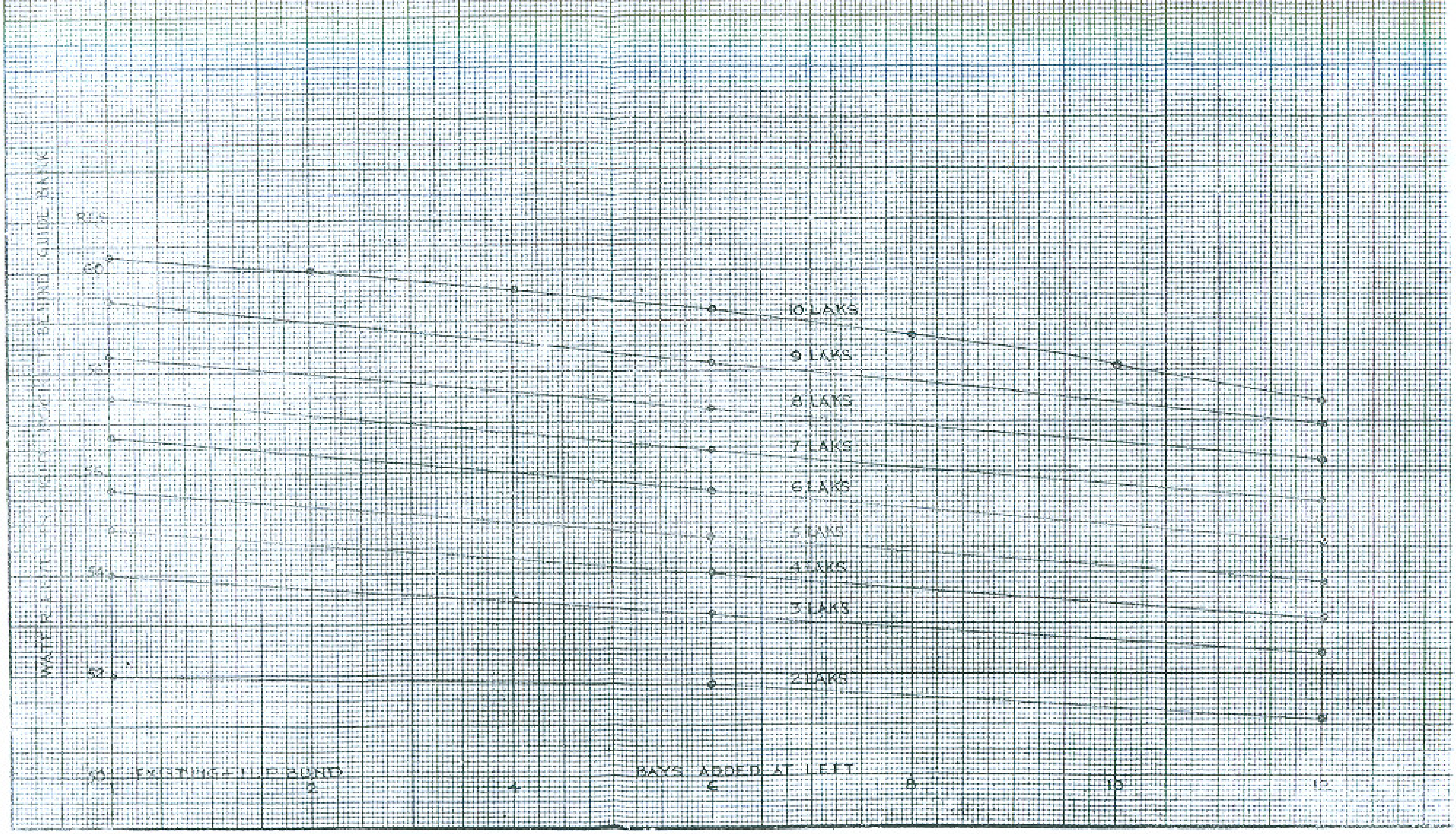
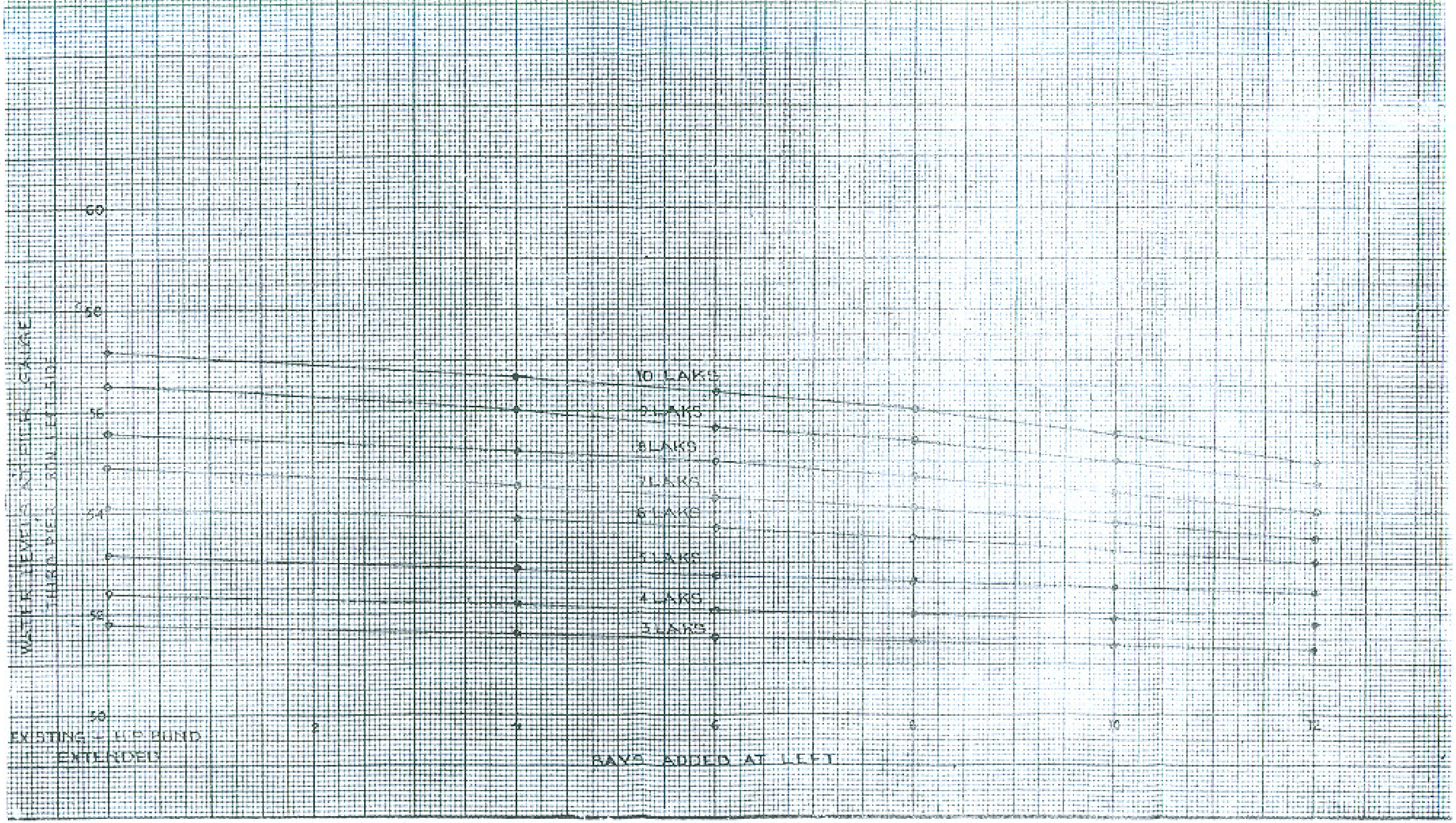
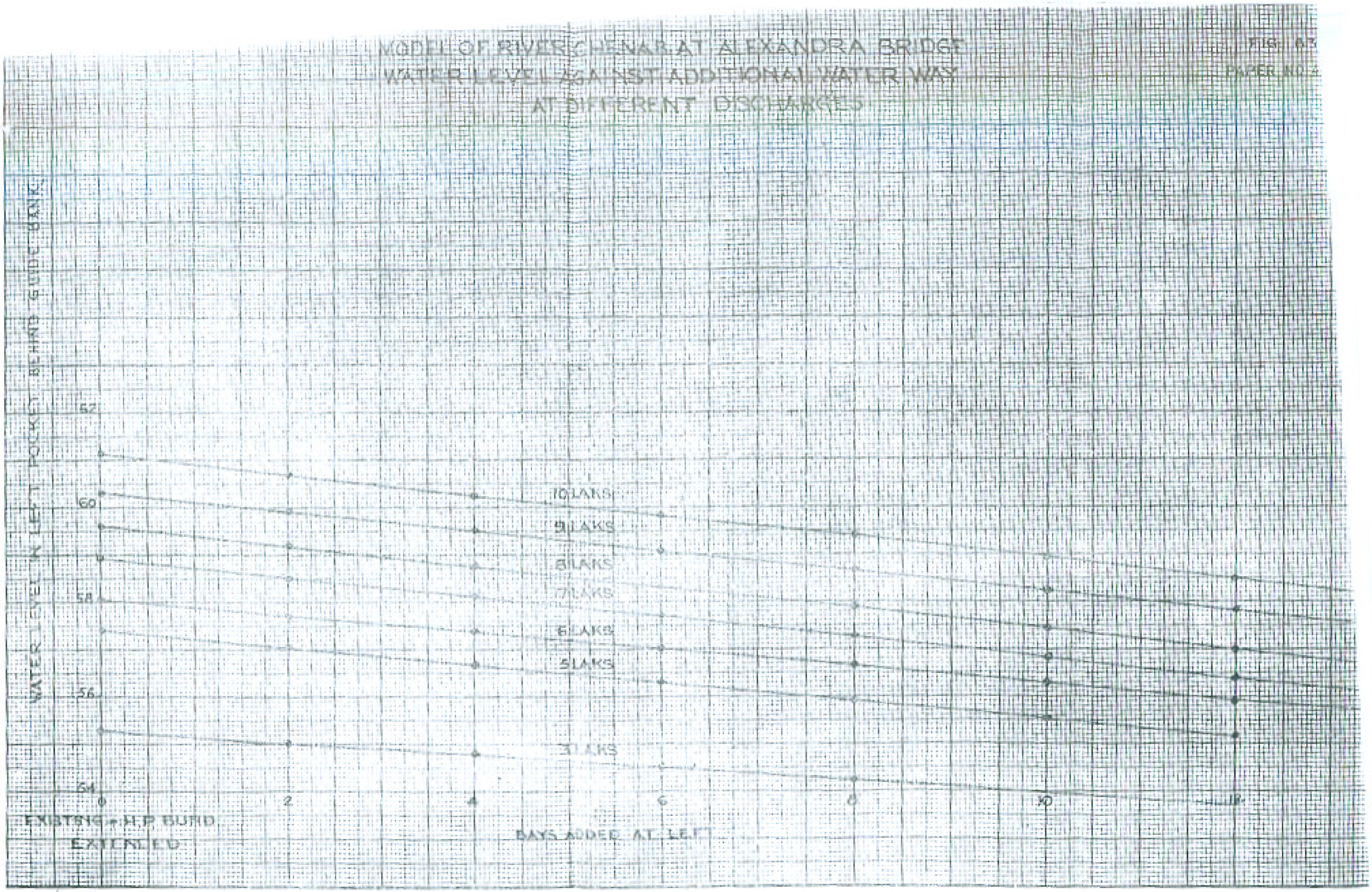


FIG. 8.2
PAPER NO 435

MODEL OF RIVER CHANAB AT ALEXANDRA BRIDGE
WATER LEVELS AGAINST ADDITIONAL WATER WAY AT DIFFERENT DISCHARGES




MODEL OF RIVER CHENAB AT ALEXANDRA BRIDGE
WATER LEVEL AGAINST ADDITIONAL WATER WAY
AT DIFFERENT DISCHARGES
WATER LEVEL IN LEFT POCKET BEHIND GUIDE BANK.
62
60
58
56
54
0
2
4
6
8
10
10 LAKS
9 LAKS
8 LAKS
7 LAKS
6 LAKS
5 LAKS
3 LAKS
EXISTING H.P BUND
EXTENDED
BAYS ADDED AT LEFT

MODEL OF RIVER CHENAB AT ALEXANDRA BRIDGE
WATER LEVEL AGAINST ADDITIONAL WATER WAY
AT DIFFERENT DISCHARGES

FIG. 8.4
PAPER NO. 42

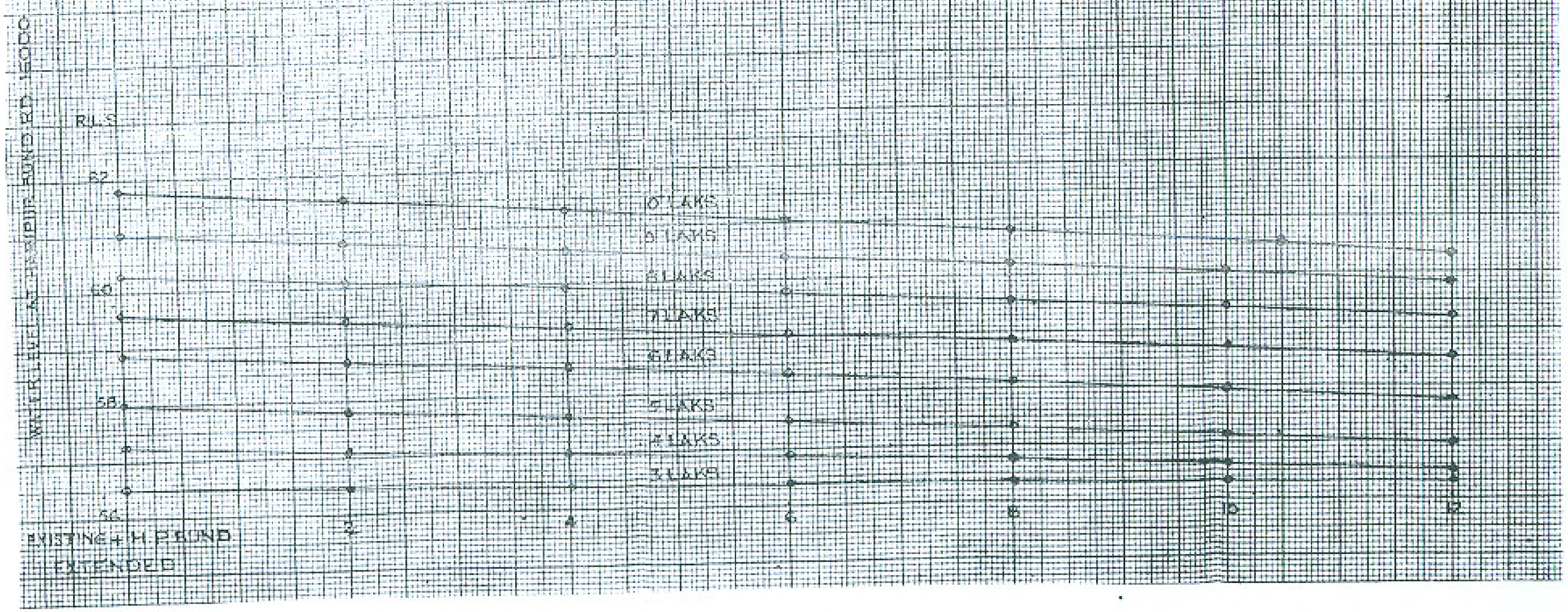
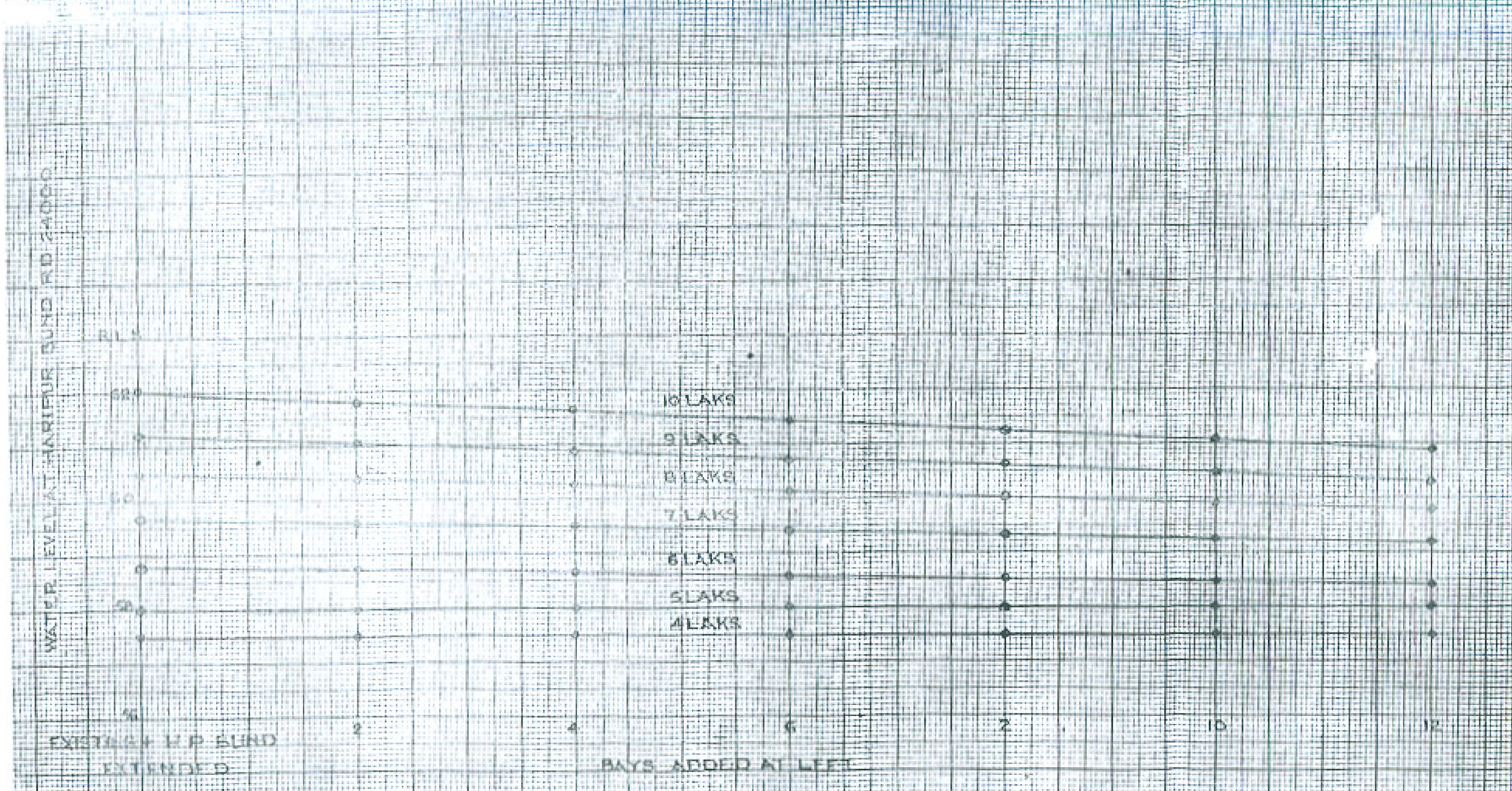
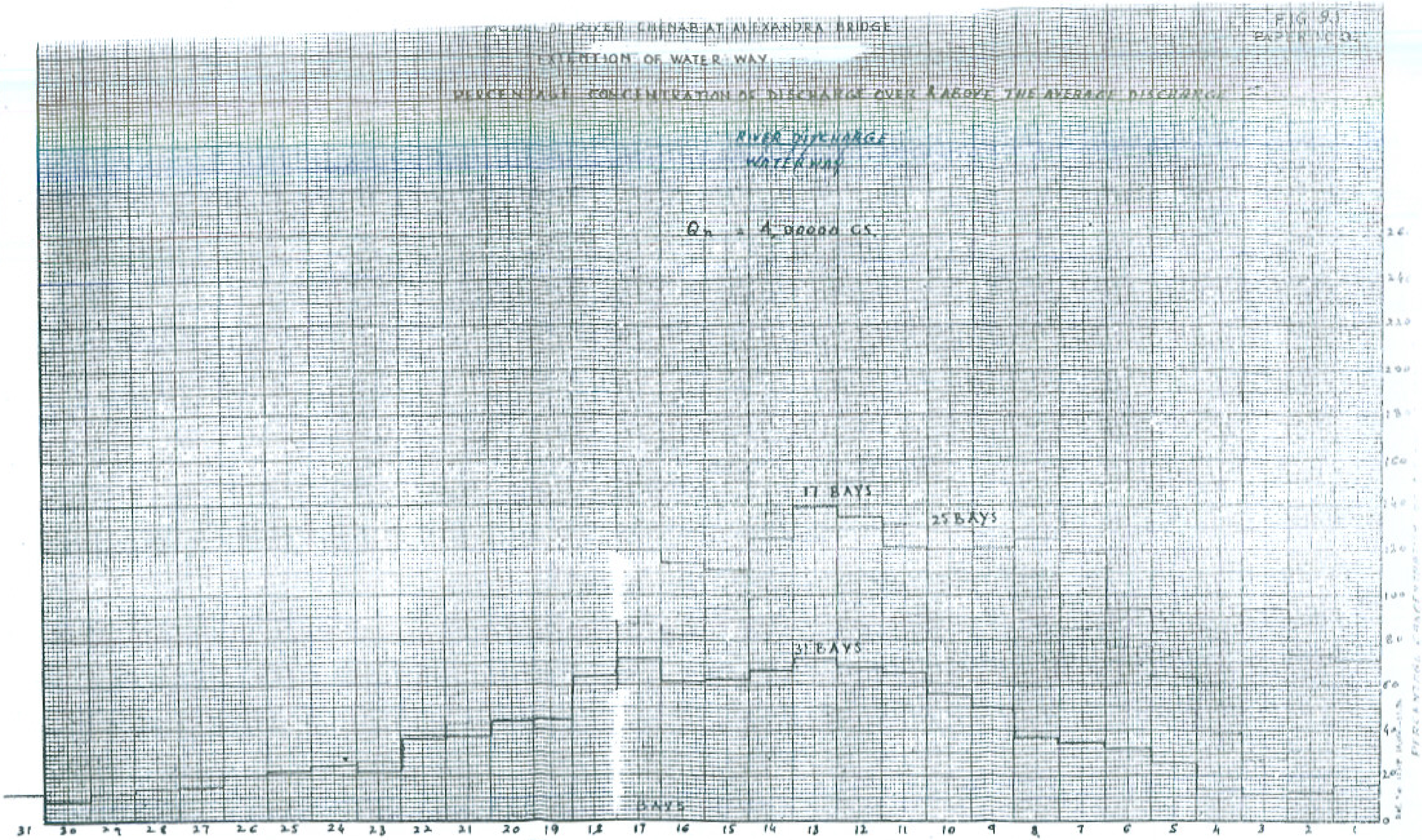
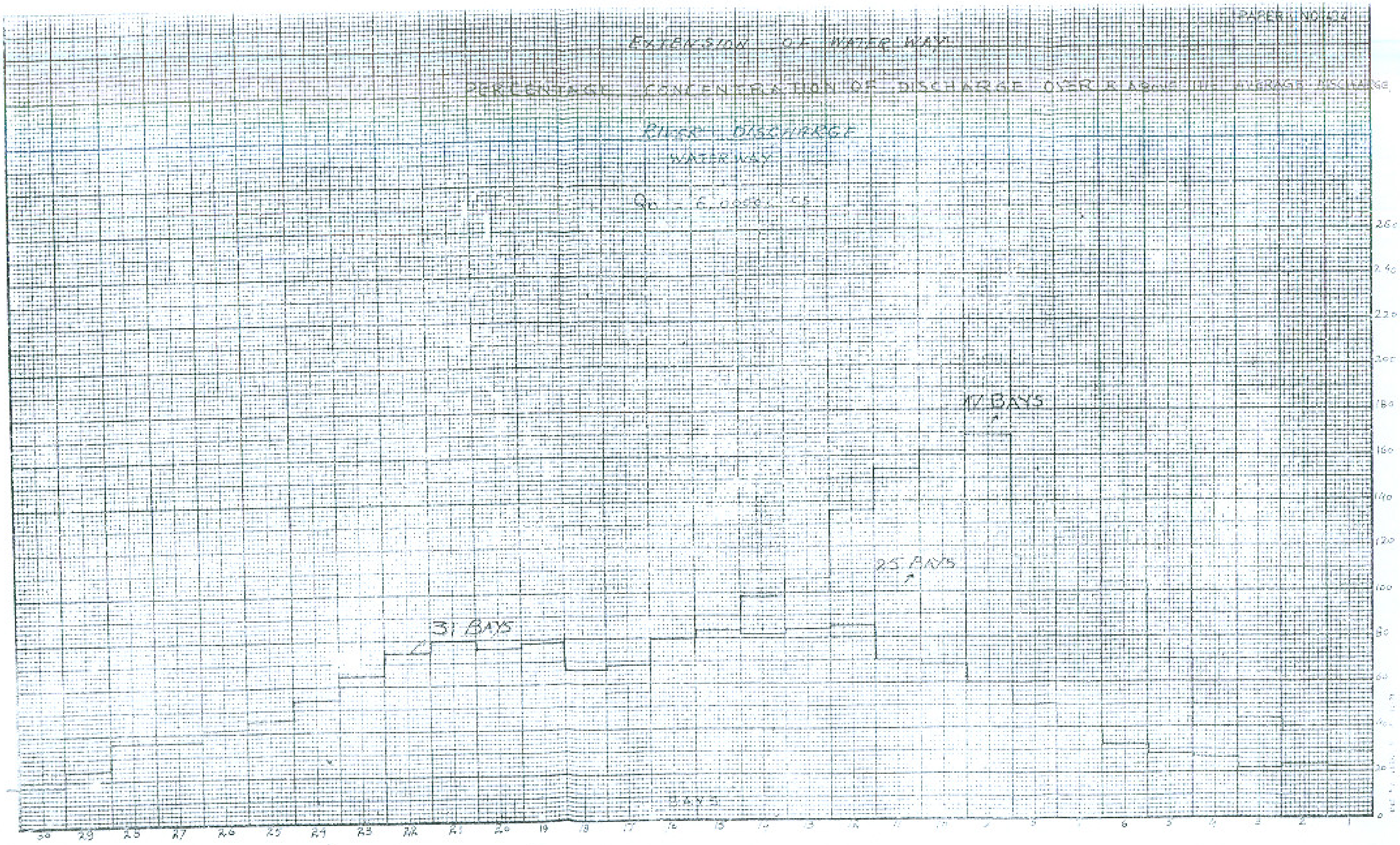


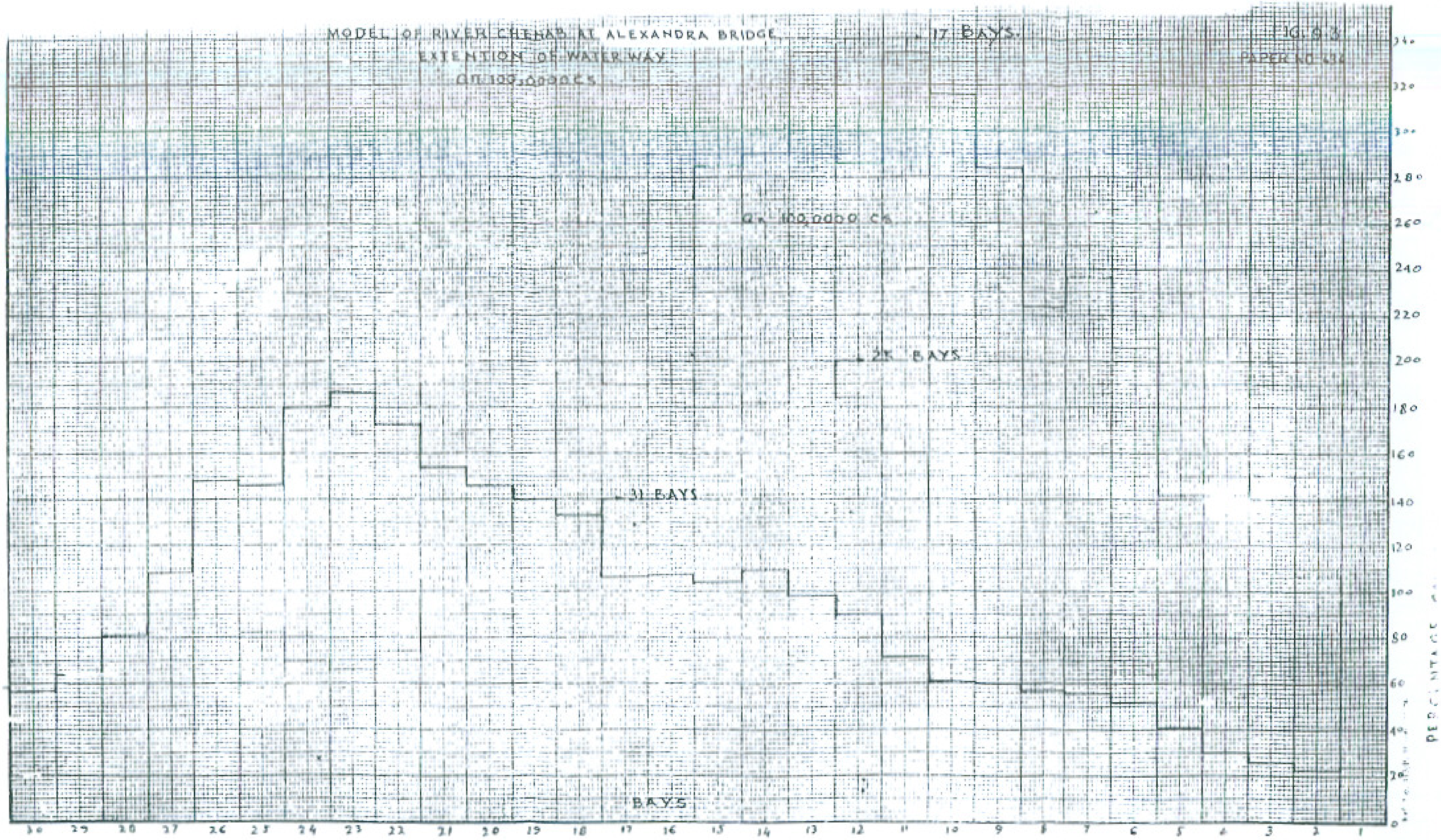
FIG. 8.5
PAPER NO. 434

MODEL OF RIVER CHENAB AT ALEXANDRA BRIDGE
WATER LEVEL AGAINST ADDITIONAL WATERWAYS
AT DIFFERENT DISCHARGES



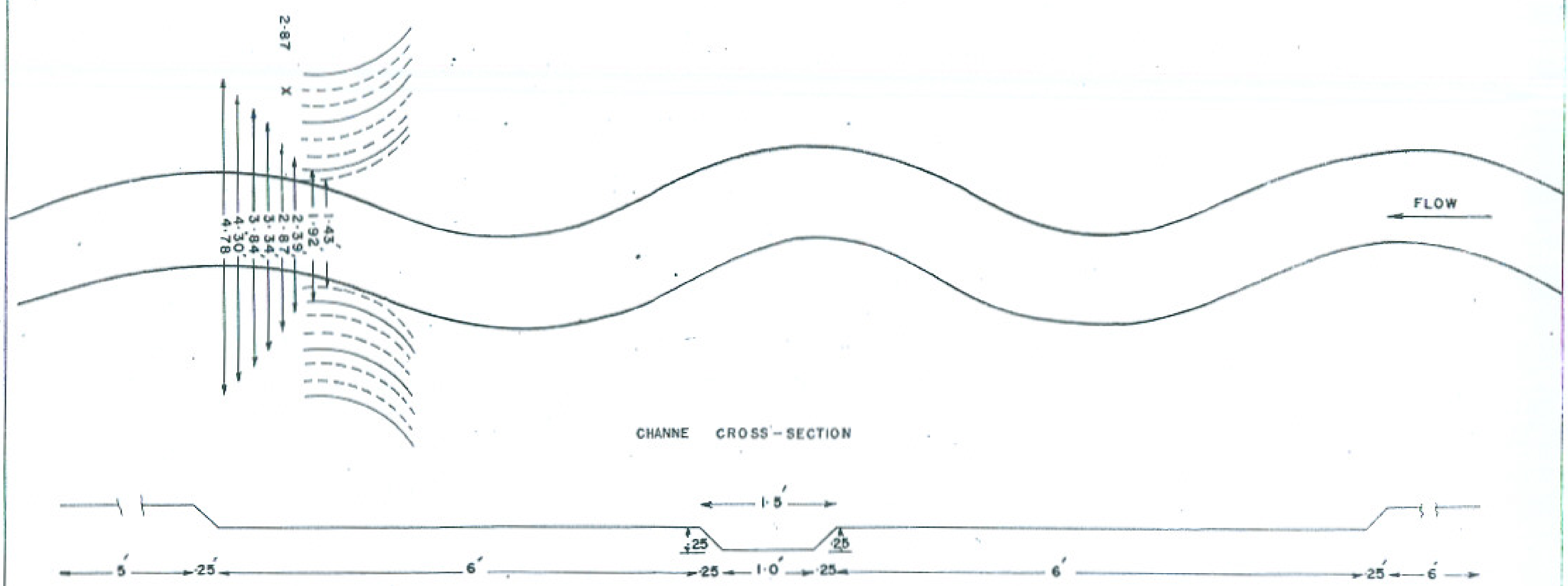

MODEL OF RIVER CHENAB AT ALEXANDRA BRIDGE
EXTENTION OF WATER WAY
PERCENTAGE CONCENTRATION OF DISCHARGE OVER & ABOVE THE AVERAGE DISCHARGE
RIVER DISCHARGE
WATER WAY
Qn = 4,00000 CS.
17 BAYS
25 BAYS
31 BAYS
BAYS
FIG 9.1
PAPER NO. 3.


PAPER NO 104
EXTENSION OF WATER WAY
PERCENTAGE CONCENTRATION OF DISCHARGE OVER & ABOVE THE AVERAGE DISCHARGE
RIVER DISCHARGE
WATER WAY
31 BAYS
25 BAYS
17 BAYS
DAYS
260
240
220
200
180
160
140
120
100
80
60
40
20
0
30
29
28
27
26
25
24
23
22
21
20
19
18
17
16
15
14
13
12
6
5
4
3
2
1



MODEL OF RIVER CHENAB AT ALEXANDRA BRIDGE
EXTENTION OF WATER WAY
Q = 100,0000 CS
FIG. 9.3
PAPER NO. 434
17 BAYS
Q = 100,0000 CS
25 BAYS
31 BAYS
BAYS
PERCENTAGE

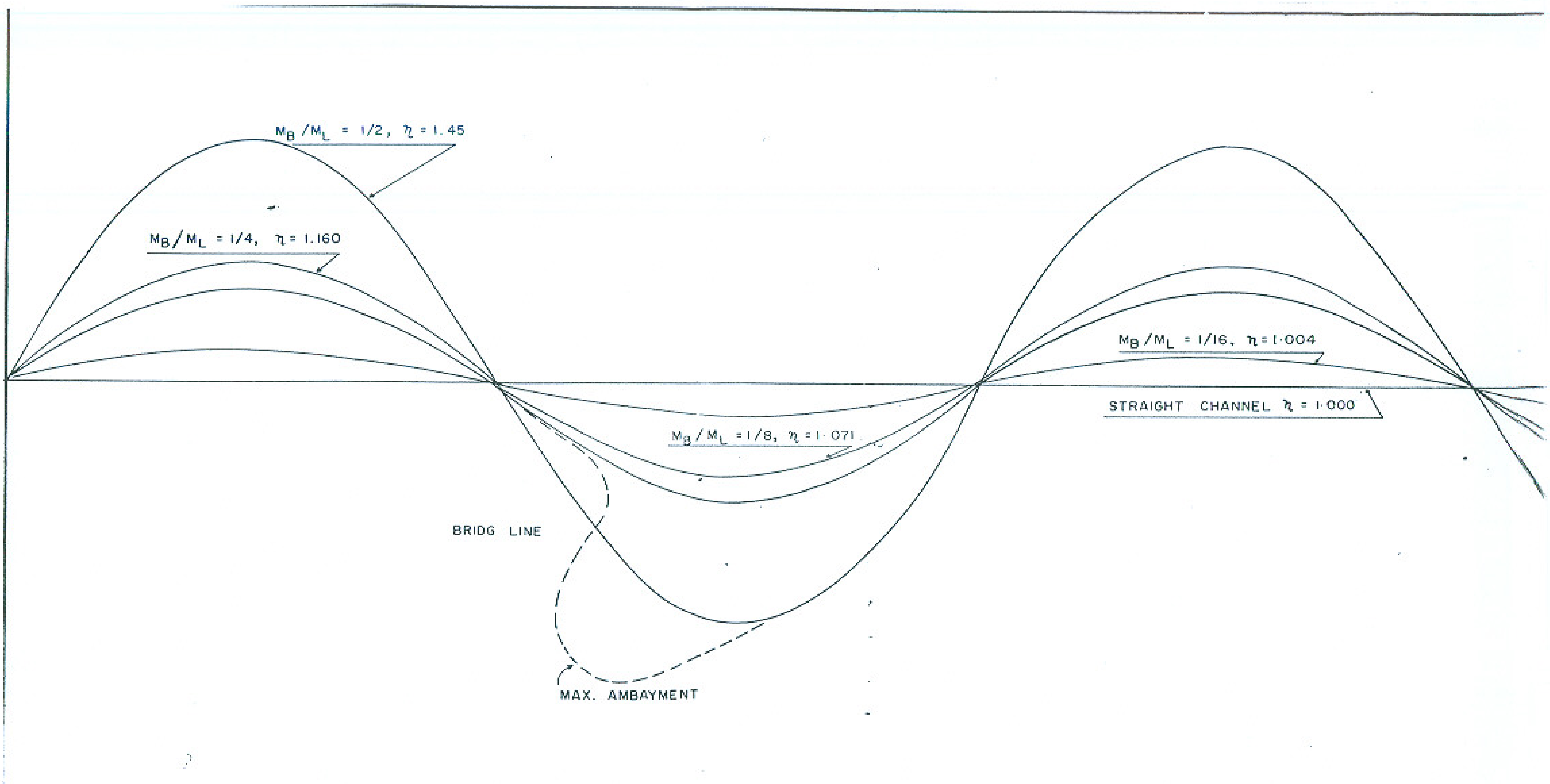
FIG: 10·1
PAPER NO 434



ORIGINAL MOULDED CHANNEL $Y = \frac{A \sin 2\pi x}{22 \cdot 4}$

$M_B / M_L = 1/16$	BRIDGE WIDTH	K
	1·43	0·6
	1·92	0·8
	2·39	1·0
	2·87	1·2
	3·34	1·4
	3·84	1·6
	4·30	1·8
	4·78 Ft.	2·0

OPTIMUM WATERWAY OF A BRIDGE



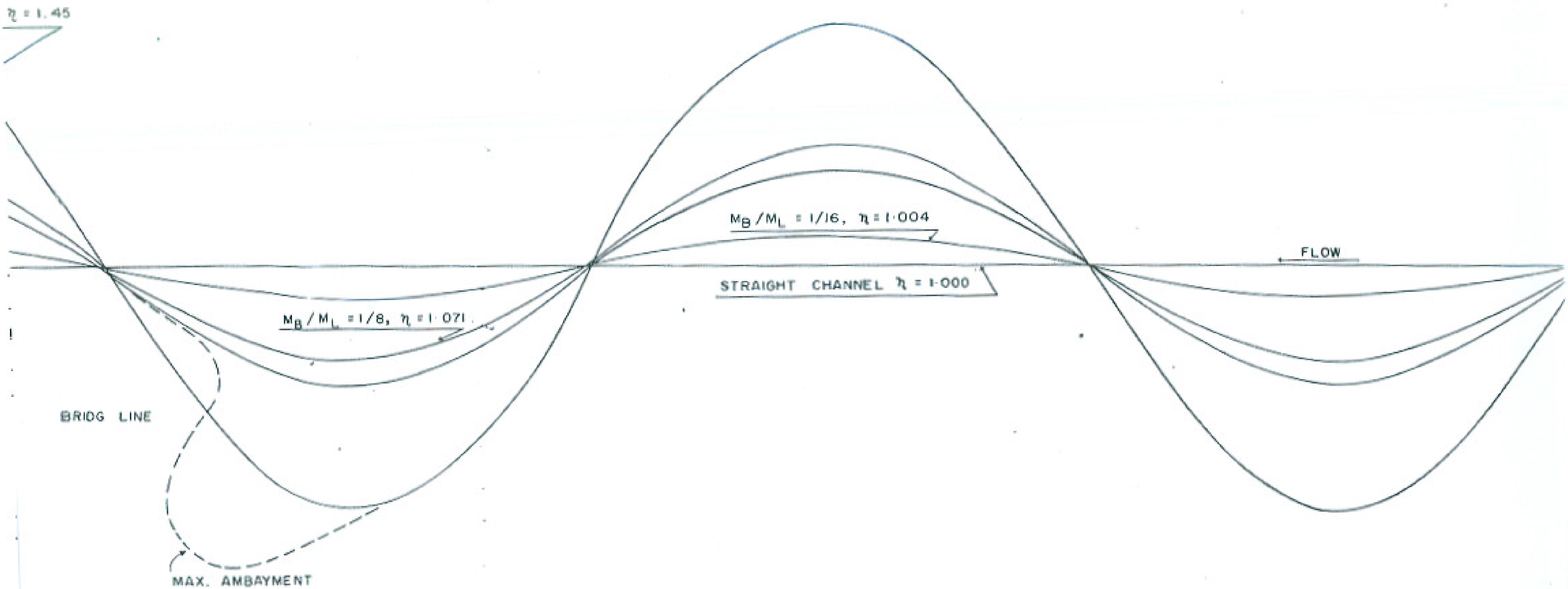
CENTRAL L
PROFILE E

0 2·5

RWAY OF A BRIDGE

FIG: 10-2

PAPER NO. 434



CENTRAL LINE OF THE VARIOUS CHANNEL PROFILES

PROFILE EQUATION : $Y = A \sin\left(\frac{2\pi x}{22 \cdot 4}\right)$

0 2·5 2·0 7·5 10·0

OPTIMUM WATERWAY OF A BRIDGE

MUNIR

Fig 10·3
PAPER NO. 434

TEST DATA

BED MATERIAL	SAND OF UNIFORM DIAMETER $d_{50} = 0{\cdot}19$ mm.
DISCHARGE	$= 0{\cdot}15$ TO $0{\cdot}8$ C.F.S.
SLOPE	$= 0{\cdot}002$

"LABORATORY STUDY OF RIVER TRAINING MEASURES"
DISCHARGE HYDROGRAPH

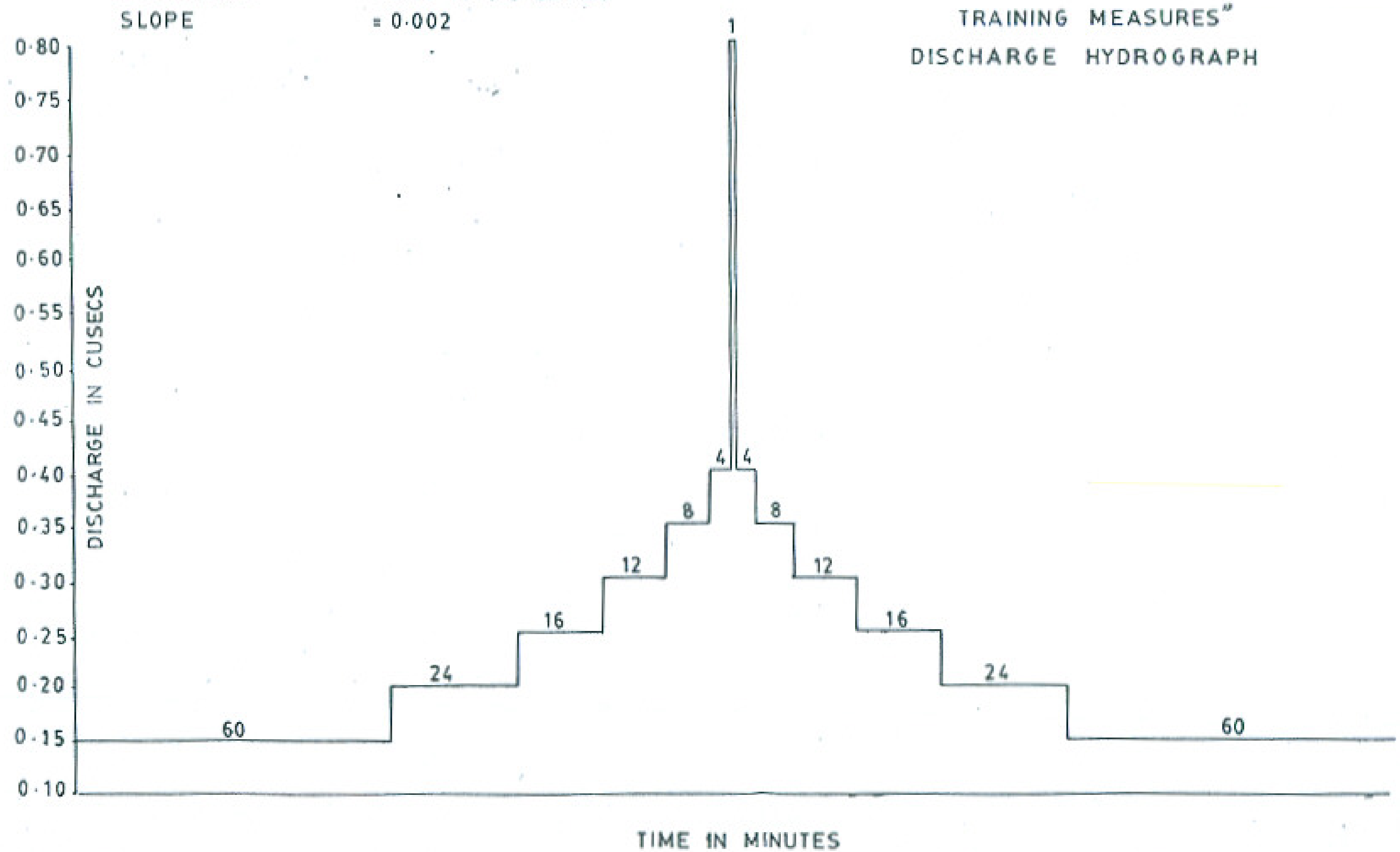
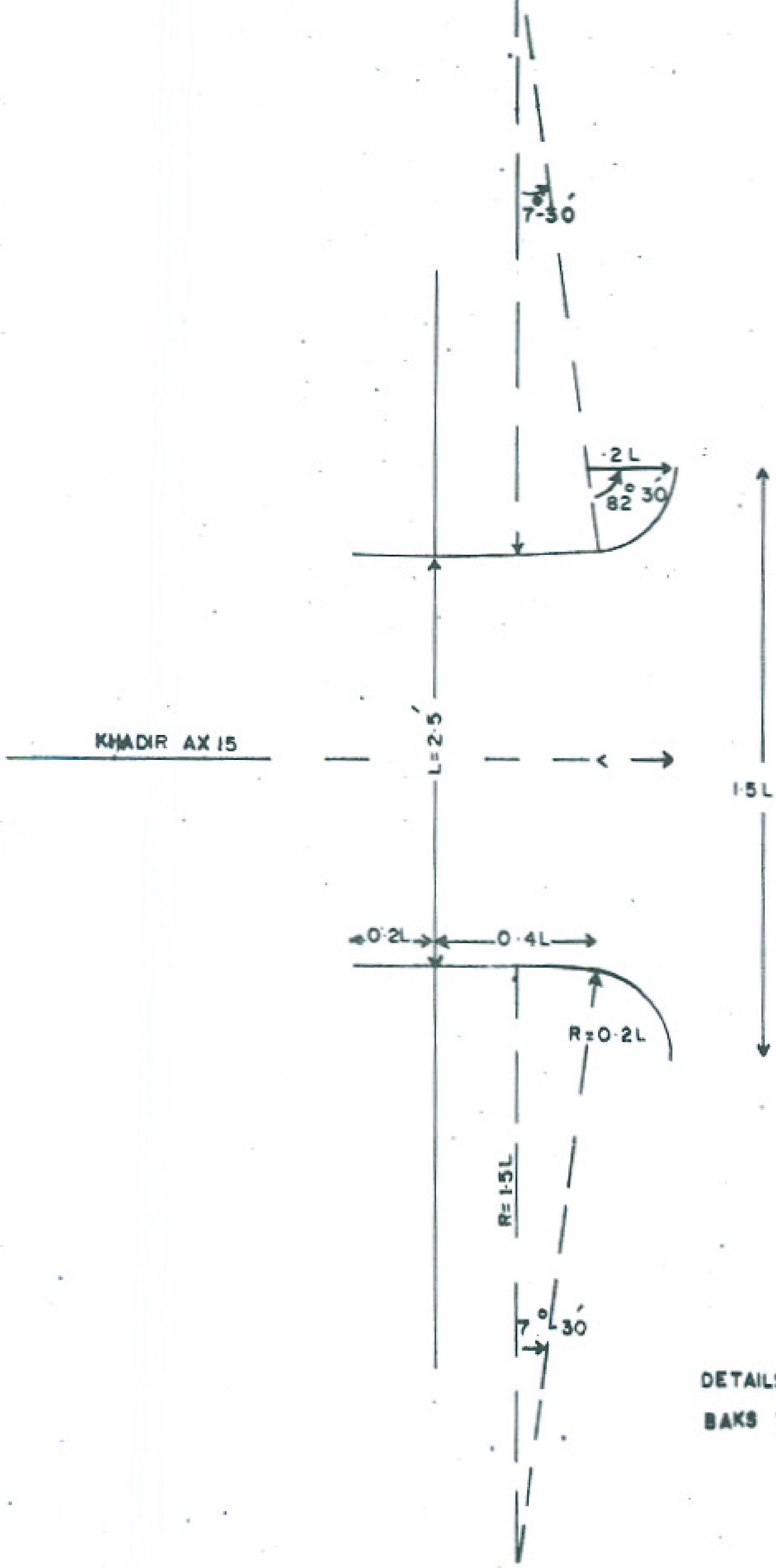


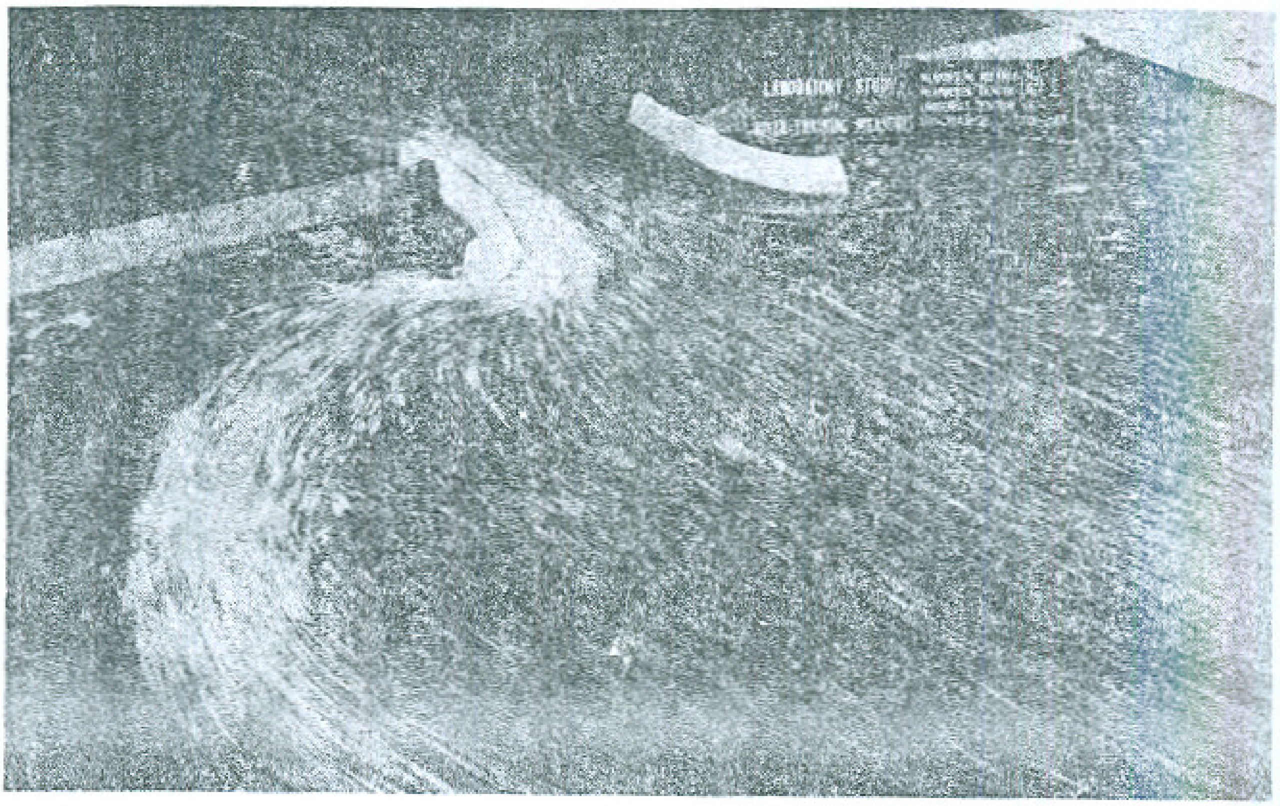
FIG: 10-4
PAPER NO 434



DETAILS OF SYMMETRICAL GUIDE BAKS WITH WATERWAY, L= 2·5'

OPTIMUM WATERWAY OF A BARIDGE

Fig. 10.5
PAPER NO 434

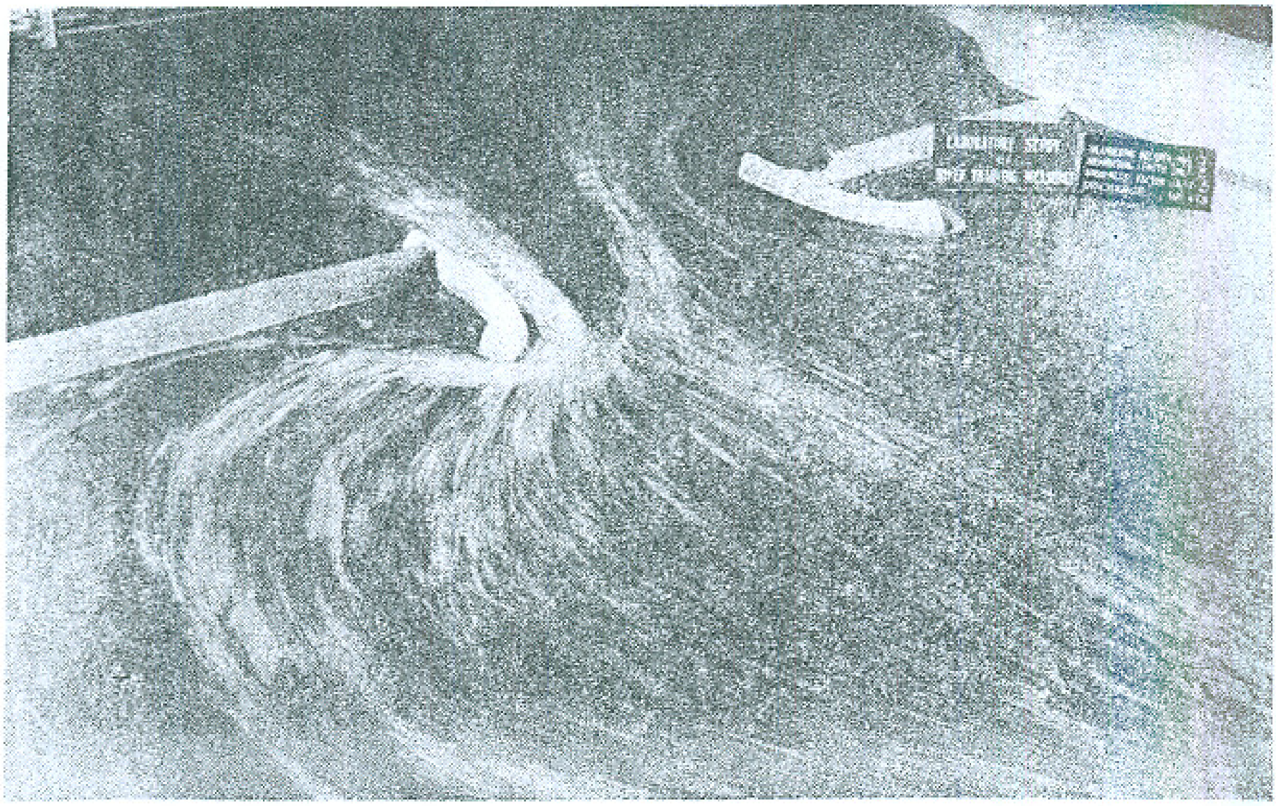


FLOW CONDITION
OPTIMUM WATERWAY OF A BRIDGE
CONCENTRATION OF FLOW AT GUIDE BANKS HEAD DUE TO OBLIQUITY OF APPROACH AT $Q = 0.8$ cs

$M_B/M_L = 1/2$

$K = 0.8$

Fig 10. 6
PAPER NO 434



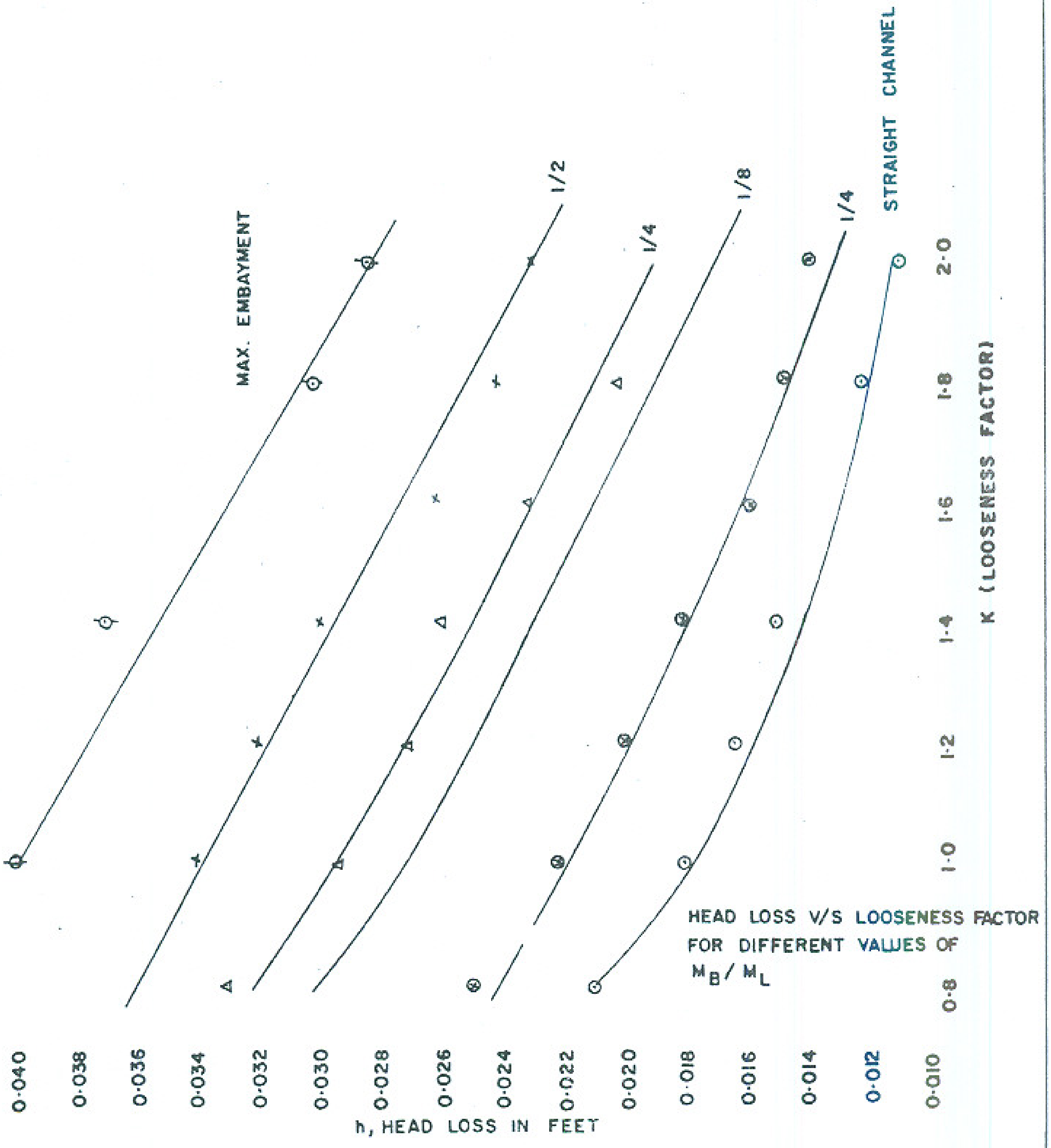
FLOW CONDITION
OPTIMUM WATERWAY OF A BRIDGE
FLOW CONDITION AT THE BRIDGE GUIDE BANKS

M_B/M_L = 1/2

K = 1. 6

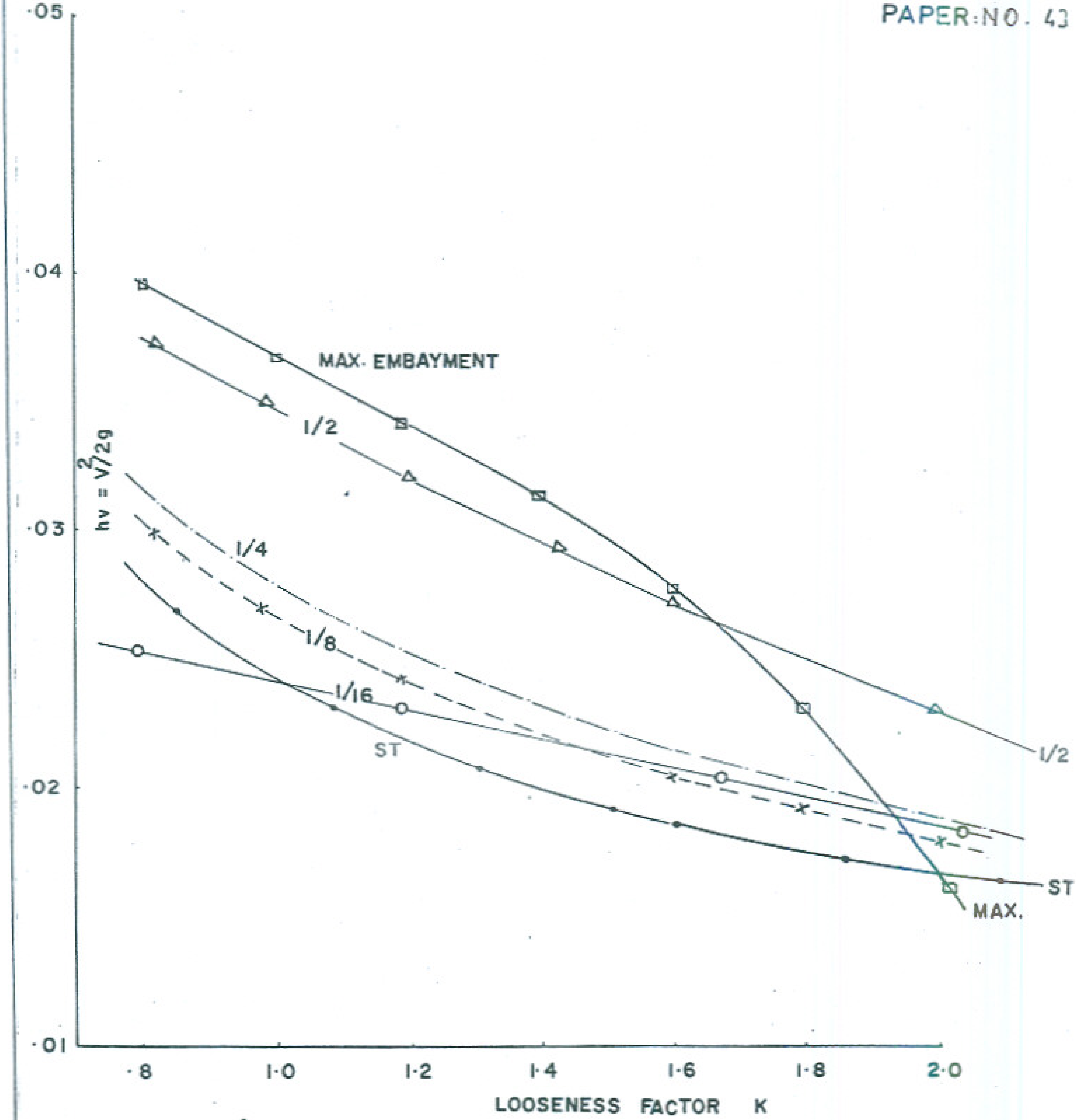
Q_* = 0. 8 cs

FIG: 10·7
PAPER NO 434



OPTIMUM WATERWAY OF A BRIDGE

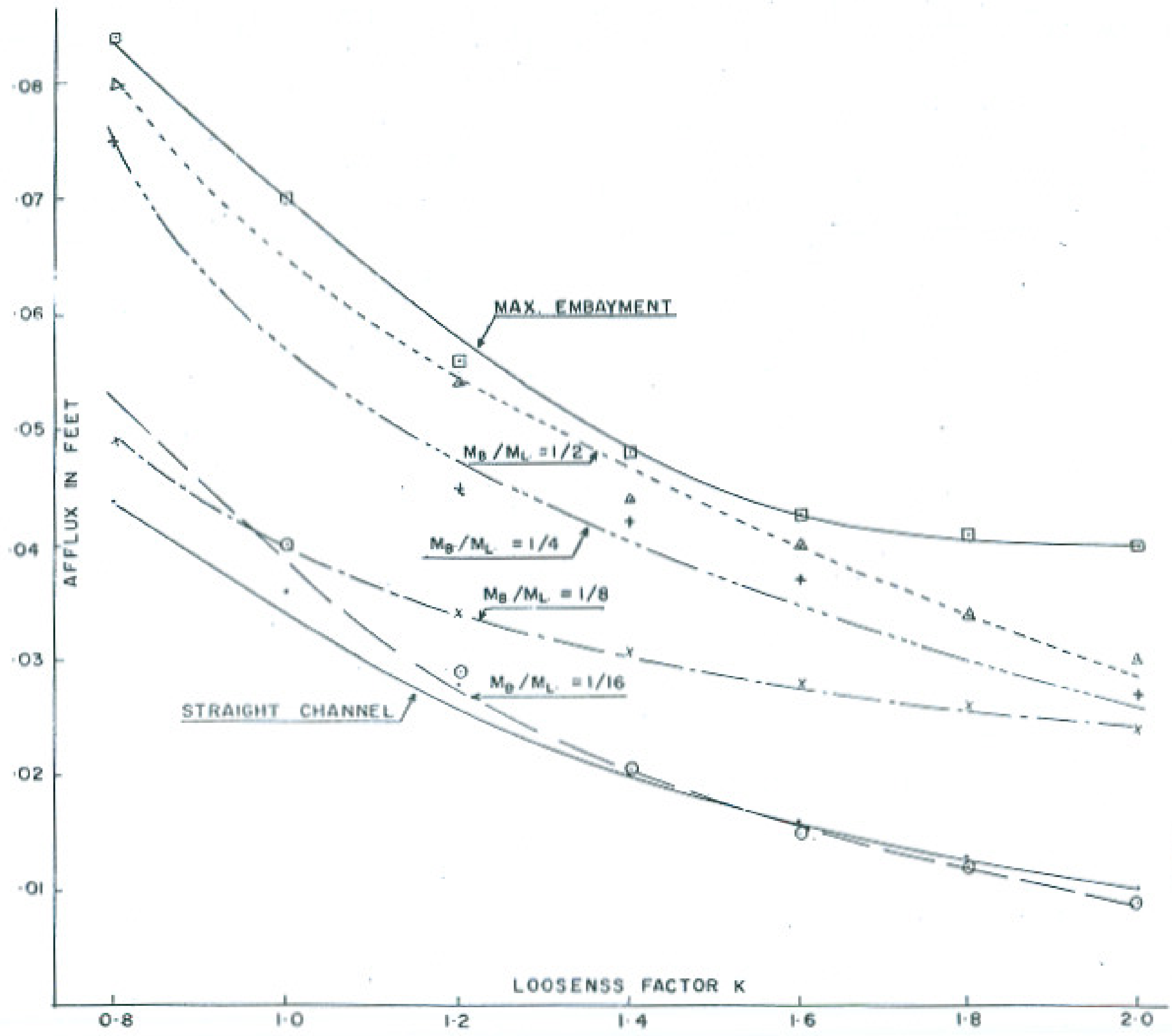
FIG: 10·
PAPER:NO. 43



VELOCITY HEAD $hv = V^2/2g$ V/S K
FOR DIFFERENT SINUOSITIES
AT Q=0·80 CS

OPTIMUM WATERWAY OF A BRIDGE

FIG: 10·9
PAPER NO 434

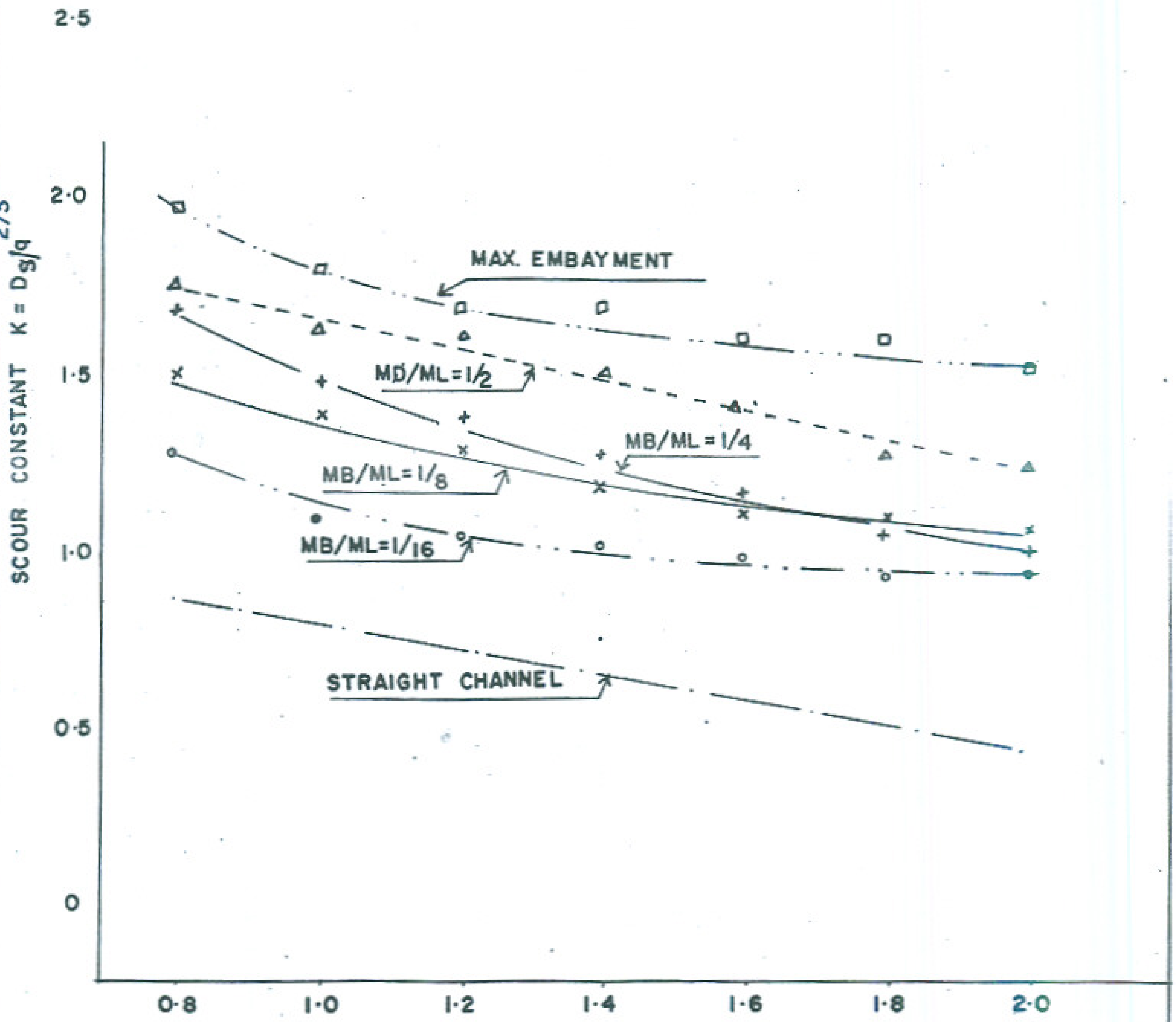


AFFLUX U/S OF A BRIDGE V/S LOOSENES FACTOR 'K' FOR DIFFERENT CHANNEL APPRAOCHES

OPTIMUM WATERWAY OF A BRIDGE

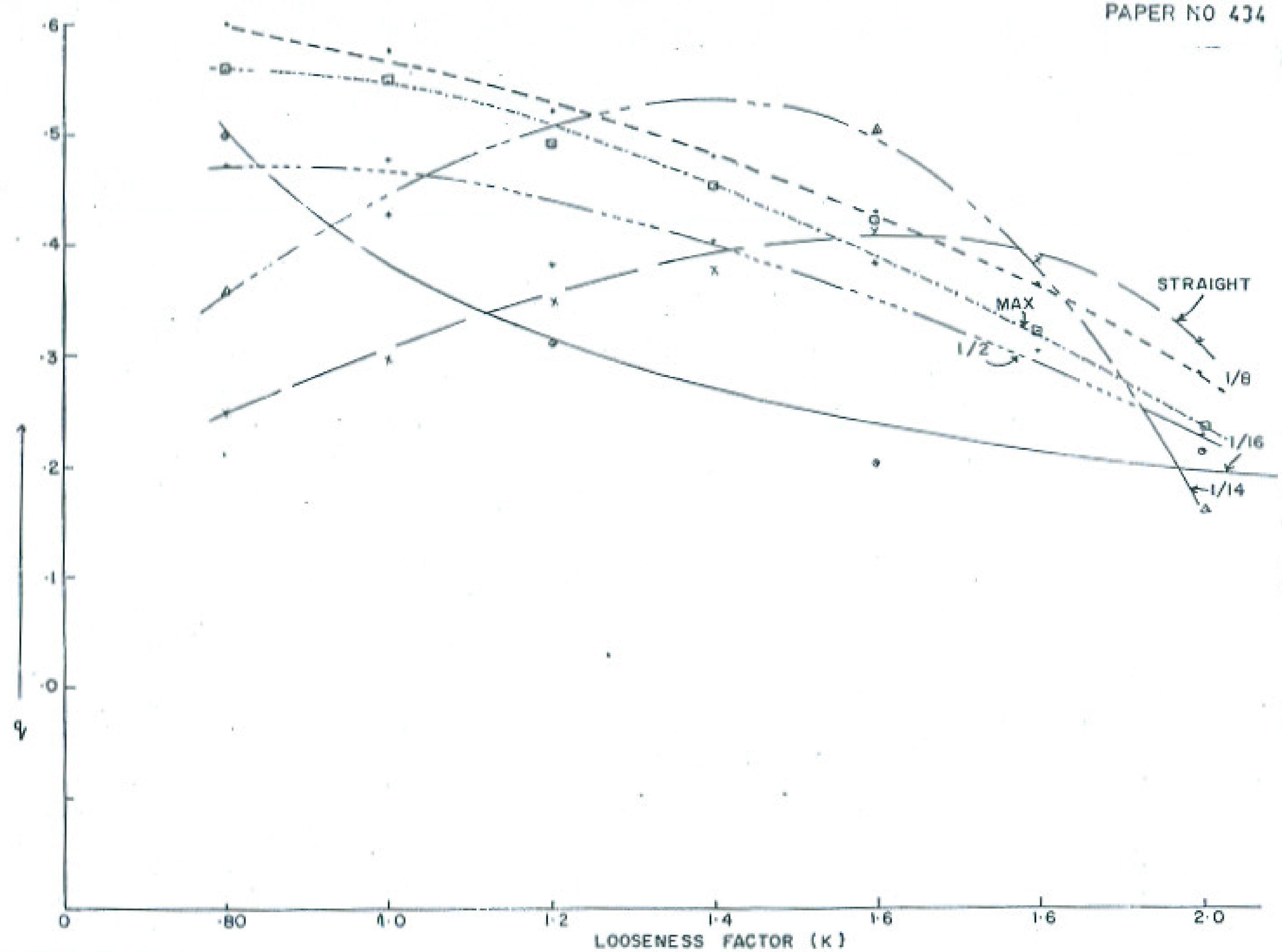
MUNIR

FIG: 10.10
PAPER NO 434



SCOUR CONSTANT K1 V/S LOOSENESS FACOK K AT THE BRIDGE

FIG 10·11
PAPER NO 434



MAX. DISCHARGE INTENSITY q V/S K LOOSENESS FACTOR WITH VARIOUS SINUOSITIES

Munir