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# Design of Alluvial Channels as influenced by Sediment Charge

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**Synopsis.**

Channels in alluvium are self-formed, in which the whole perimeter is actively taking part in the flow. It is easy to assume that in such channels width, depth and slope are all variables and sediment transport makes the problem more complex. An appreciation of regime relations as developed in Indo-Pakistan sub-continent in the last four decades, is given and examined in relation to mean silt charge and silt diameter. New equations have been developed, that determine V, D and S in relation to silt charge for all types of regimes of flow *i.e.*, channels carrying 1-p.p.m to 40,000 p.m. or 1/1000 gr. litre to 40 gr. per litre.

For practical application, solved examples have been given to design seepage drains, Irrigation channels and ejector escapes.

## 1. Introduction.

As a part of Indus Water Treaty, seven new link canals varying in capacity from 6,500 to 21,700 cusecs designed to a flat gradient of 1/8000 to 1/10,500 will be constructed. Basically these have been designed on the regime relationships as formulated by G. Lacey (1) :—

$$S \propto 1/Q^{1/6}$$

$$\text{and } VS = 0.43 \times 10^{-3} f^2$$

$$\text{where } f = 0{\cdot}75\, V^2/R$$

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Lacey's silt factor f has been adopted arbitrarily.

Such large withdrawals in addition to existing off-taking channels will increase the water surface slopes at the headworks from the pond or any other control structure to the pocket in front of the intake regulator, consequently the fluid turbulance as well as sediment movement will increase comparatively and it will upset the working of the existing de-silting devices. Examples can be quoted namely :—

(1) Marala Ravi Link.

(2) Mailsi Canal, fig. 1 (a).

(3) Sukkur Canals ... Right bank canals.**

... Left bank canals.

M. R. Link in the first 20 miles has silted up on the average by 4 ft. above its designed bed as a result of increased turbulance near its Headworks. Mailsi canal silted up due to changes in the river regime and bank erosion. It will be seen from fig. 1 (a) that it took 70 days to silt up a scoured channel by as much as 2 ft. above its designed bed. In the case of Sukkur canals, Rohri canal on the left and Rice canal on the right bank have almost equal full supply discharges and bed widths ; both were designed in 1942 with nearly the same slope of 1/12,000 in their headreach. Right bank canals silted up and created a slope of 1/7300, while left bank canal heavily retrograded its bed and re-adjusted its slope to 1/19,800. Of course the Rice canal is non-perenial while Rohri canal is perenial. It shows that if excessive silt charge enters into a canal, steeper slopes will be required and for this a higher pond level at the headworks may become necessary. As such it is essential that the river approach and water surface slopes are so controlled that the regimes of flow in the river around the Headworks and the off-taking canals are not changed, otherwise the frequency of remodelling of Headworks will increase. It is quite likely that recurring expenditure will be more and more and monetary returns become less and less than it used to be in the past.

Previously, the authors[2] examined silt data of L. C. C. channels and Laursen's flume data and derived functions, relating silt intensity C for coarse and medium sand in grams per litre with hydraulic parameters in the following forms :—

(1) $C = \phi\ (V^2 S)$.

(2) $C = \phi\ (V^3 / R)$.

This study was for a limited range and moreover two equations were insufficient for design purposes. In this paper the authors have widened the scope of study from a lower regime of a plane bed when there is hardly any silt movement, to a higher regime of flow when as much as 10 to 30 grms./litre of silt is carried by a stream. Our own data for the

period 1940, 1942 and 1943 of L. C. C. stable channels, discharge varying between 40 to 10,000 cusecs, data from river Ravi (Q = 10,000 to 100,000 cusecs) and experimental flume data of well known hydraulic engineers such as Albertson, Simmons, Richardson, Laursen, Vanoni and Allen as published in the various journals of American Society of Civil Engineers and other publications have been utilised in obtaining two other functions, one containing silt intensity with discharge, slope and bed sand and the other between shear stress and silt charge. Based on these two functions, the silt carrying capacities of the existing channels can be calculated and new channels designed from a new approach. Silt carrying capacity of the designed link channels has been calculated as there is no scope of revising their slopes. Silt entry into these channels in excess of their capacities will have to be ejected by suitable methods and silt carried back to river through sluices or ejector escapes. These small ejector escape channels should be sufficiently steep to carry the flow that is almost 10 times as much concentrated in silt as the larger link channel. Here again the derived equations will be helpful in designing the ejector escape.

Moreover slope / discharge relations of Lacey and Bose have been examined and the divergence of their coefficients in relation to silt charge has been studied.

The object of this paper is to stress the role of suspended silt charge in relation to regime concept for designing new channels and to present new methods and functional relations based on model and proto-type data.

## II. Development of Regime formulae for artificial channels.

Silting and scouring of Irrigation channels baffled all research engineers of 19th century, till Kennedy in 1896 after studying the behaviour of U.B.D.C. system produced his classical relation for velocity depth / for non-scouring non-silting channels. His equation is :

$$V = 0.84\ D^{0.64} \qquad \text{.....(1)}$$

Later on it was found that the coefficient and the index power may vary for different systems. In 1919, Lindley introduced the shape factor of channels from data of Lower Chenab Canal system.

$$V = .95\ D^{0.57} \qquad \text{.....(2)}$$

$$V = 0.57\ B^{0.355} \qquad \text{.....(3)}$$

$$B = 3.80\ D^{1.61} \qquad \text{.....(4)}$$

Width, depth and slope were observed ; and mean velocity was calculated by using Kutters equation, adopting n = .0225.

In 1927, Lacey presented his equations in which D was changed to R.

$$P = 2{\cdot}67 \quad Q^{\frac{1}{2}} \qquad \ldots\ldots\ldots(5)$$

$$V = 1{\cdot}15\sqrt{fR} \qquad \ldots\ldots\ldots(6)$$

$$Af^2 = 4V^5 \qquad \ldots\ldots\ldots(7)$$

$$R = 0{\cdot}47\,(Q/f)^{\frac{1}{6}} \qquad \ldots\ldots\ldots(8)$$

$$S = \frac{f^{\frac{5}{3}}}{1830} \cdot \frac{1}{Q^{\frac{1}{6}}} \qquad \ldots\ldots\ldots(9)$$

$$S^* = 0{\cdot}39\, f^{\frac{5}{3}} / q^{\frac{1}{3}} \qquad \ldots\ldots\ldots(10)$$

$$V = 16\ R^{2/3}.S^{1/3} \qquad \ldots\ldots\ldots(11)$$

and f was defined as $0{\cdot}75\ V^2/R$

It will be seen that Lacey's f assumes the form of Froude No. $\frac{V}{\sqrt{gD}}$ such that $f = 24\,(F)^2$.

Lacey assumed that similar channels will have the same silt factor 'f'. With the aid of experiments conducted in Poona by Inglis, f was related to mean bed silt size as:

$f = 1{\cdot}75\sqrt{m}$ where m is mean diameter in m/m,

This suggests that the size of the bed mateiral fixes Lacey's f on which the design of the channel is based. But for the same velocity and discharge in two channels of different Lacey's f, the relations show that:

$$V = 1.15\sqrt{fR}$$

$$S^* = 0{\cdot}39\,\frac{f^{5/3}}{q^{1/3}}$$

$$\frac{V1}{V2} = \sqrt{\frac{f_1R_1}{f_2R_2}}$$

$$\frac{S_1}{S_2} = (f_1/f_2)^{5/3}$$

when $f_1 < f_2$

$R_1/R_2 = f_2/f_1 > 1$

and $S_1/S_2 = (f_1/f_2)^{5/3} < 1$

It shows that with lower values of Lacey 'f', the channels of equal discharge are deeper and flatter in gradient as compared to channels of

higher 'f'. Though Lacey does not include the effect of charge in his relations, but 'f' probably incorporates some average value of silt charge. It will not be incorrect to assume that channels passing excessive sediment will be of shallow depth and steeper in gradient. It that case Laceys

$$f_{(RS)} = 192 R^{1/3} S^{2/3} \quad \text{.....(12)}$$

becomes indicative of silt charge as well.

Because some channels showed divergence from Lacey's original velocity/depth relation, Inglis[3] introduced $f_{(VR)}$, $f_{(RS)}$ in the general equation :

$$V = 16{\cdot}02\, R^{2/3} S^{1/3} \cdot (f_{VR} \cdot f_{RS})^{1/2} \quad \text{.....(13)}$$

Inglis presented a new set of formulae that included the effect of charge, but the constants of equations were not determined. In 1935. Bose[4], after painstaking collection and statistical analysis of the data of L.C.C. stable channels, presented his equations :

$$P = 2{\cdot}8\, Q^{1/2} \quad \text{.....(14)}$$

$$S \times 10^3 = 2{\cdot}09\, d^{\cdot 86}/Q^{0{\cdot}21} \quad \text{.....(15)}$$

$$R = 0{\cdot}47\, Q^{1/3} \quad \text{.....(16)}$$

In 1951, Blench[5] presented his concept of bed and side factor and modified Lacey relations but still omitted silt charge.

### III. Divergence of Slope Coefficient related to silt transport.

Slope relations of Lacey and Bose were checked with the data of stable channels in the Irrigation Research Lahore, but divergence existed to as much as $\pm$ (10 to 20)%. Lacey introduced his shock theory for explaining the divergence. The authors consider that the divergence is due to the silt charge. These equations have been examined afresh with regard to the silt charge as observed during the years 1940, 1942 and 1943. The data of other large channels also has been used for comparison of the coefficient in the two slope equations :—

Lacey equation No. (11).

(*i*) $$\frac{V}{R^{2/3} S^{1/3}} = 16{\cdot}02 = K_1$$

(*ii*) Bose equation : (15)

$$\frac{S \cdot Q^{0 \cdot 21}}{m^{\cdot 86}} = 2 \cdot 09 = K_2$$

are plotted in curves Fig. 2 (a, b).

In curve fig. 2 (a), the Lacey's coef: $K_1$ varies from 13·3 to 18·3 for silt charges 0·2 to 0·7 gr./litre or 200 to 700 p.p.m. The divergence from the mean value of 16·02 is:

$$\frac{2 \cdot 68}{16 \cdot 02} = -16 \cdot 80\%$$

$$\frac{2 \cdot 3}{16 \cdot 03} = +14 \cdot 1\%$$

of an order of about −17 to +14%. It is derived from this plotting that Lacey coef. of $K_1 = 16.02$ may be applicable to a channel carrying silt load of 0·33 gr./litre.

In fig. 2 (b) Bose coef. in slope discharge relation, shows a similar divergence. The value of $K_2$ varies from 1·75 to 2·85. The percentage divergance is calculated as below:—

$$-\frac{\cdot 34}{2 \cdot 09} = -16 \cdot 4\%$$

$$+\frac{\cdot 76}{2 \cdot 09} = +36 \cdot 5\%$$

Mean value of Bose coefficient $K_2 = 2 \cdot 09$ may be applicable to a channel carrying silt charge of about 0·3 gr/litre, almost the same as for Lacey's general equation. The divergence shown by the Bose relation is higher than Lacey relation. From the plottings, it is obvious that divergence from the mean velocity is most probably due to silt charge:

$$K_1 \text{ and } K_2 \propto C$$
$$\propto 1/m.$$

## IV. Development of new regime equation with regard to the silt carrying capacities of the channels.

As already stated in the introduction that field data of stable L.C.C., other channels and Laursen's (6) flume data were examined for their silt carrying capacities, and equations developed previously are given below:—

Silt intensity $C = \phi \left( \frac{}{V/R} \right)$ is plotted in fig. 3;

another function is

$$C = \phi \left[ V^{2} \cdot S \right]$$

The equations found valid for silt charge upto 2 gr/litre were :

(i) $\overline{C} = 0.1 + 0.1(V^3 / R - 1.5)^{4/3}$ ..... (17)

(ii) $\overline{C} = \left[ 8g.V^2 .S \right]^{1.25}$ ..... (18)

applying Lacey's equation.

$$VS = 0.4 \times 10,^{-3} f^{2.}$$ ..... (19)

equation (18) reduces to :—

$$C = 0.1 (f^2 V)^z$$ ..... (20)

The index value was found to be nearly given by

$Z = \frac{\sqrt{gDS}}{W}$ = 1 to 1·7 } where $\sqrt{gDS}$ = Bed shear and W = fall velocity.

The above relations were checked with flume data of Wallingford Laboratory (7) (fig. 3). The data when plotted on our curves, showed lesser value of C for $V^3/R$ in the range of 3 to 7 tested by them. To determine the reasons for the higher value of C observed in our field data, the silt observations of coarse, medium and fine were plotted for all the observation sites on the L.C.C. channels. Observations of two such sites Burala and Lower Gugera for the years 1940, 1942 & 1943 ; are plotted in figs. 4 (a, b). In Burala Branch, winter and summer discharges vary in range of 1200 to 1800 cusecs while at Gugera, the range is 1800 to 2200 cusecs. It will be observed that in both the cases, silt charge of fine grade increases tremendously during June to September, to as much as 1.5 gr/litre due to surface erosion by rivers during floods. Coarse and medium silt also increase by almost 100% of the mean values. Similar conditions were observed for all other sites. In Wallingford Laboratory flume tests, fine silt is only 5% of the total, while in our channels fine part is excessive. This suggests that probably the presence of sufficient quantities of very fine silt increases the capacity of silt transport of the channel.

## V. Corelation of Laboratory and Flume date.

Published flume data of well known hydraulic engineers namely Simons (8, 9), Richardson (10), Laursen, Vanoni (11) and Allen and field data of L.C.C. and other channels and river Ravi has been plotted in fig. 5 (a to g) against different variable functions such as :

(i) $C = \phi (V^3/D)$ Fig. 5 (a)

(ii) $C = \phi (8gV^2S)$ Fig. 5 (b)

(iii) $C = \phi (VS)$ Fig. 5 (c)

(iv) $C = \bar{\tau} / (\gamma - \gamma_1) d$ Fig. 5 (d)

(v) $C = q^{2/3} . S$ Fig. 5 (e)

(iv) $C = q^{2/3} S/W$ Fig. 5 (f)

(vii) $C = q^{2/3} S/W^{1/2}$ Fig. 5 (g)

Field data for stable channels, river Ravi and basic flume data for .04, .1, 0.28 and 0.45 m/m are given in tables I to III and remarks are offered on each functional relation as below :—

**First Relation.**

$$\overline{C} = \phi \ (V^3 / D)$$ Fig. 5 (a)

Generally for the same value of $V^3/D$, the silt intensity is shown higher for finer grade. There is a wide divergence. Field data from L. C. C. canals finds a place intermediate of the extreme limits.

**Second Relation.**

$$\overline{C} = \phi \ (8g \ V^2 \ S)$$ Fig. 5 (b)

The data shows similar trend as for relation I above.

**Third Relation.**

$$\overline{C} = \phi \ (VS)$$ Fig. 5 (c)

Our field data is confined to mostly.

$VS = 0.4$ to $0.5$

while the range of silt charge varies from 0.2 to 0.9 gr/litre. Therefore $C = \phi$ (VS) does not appear to be a suitable function for predicting silt intensity for lower ranges. For higher regimes it behaves similarly as relation I above.

Introducing tractive force in terms of $\bar{\tau}$ and diameter, we have.

**Fourth Relation.**

$$\overline{C} = \phi \left[ \bar{\tau} / (\gamma - \gamma_1) . d \right]$$ Fig. 5 (d)

The trend is more regular probably due to silt dia 'd' in ft. introduced into the function. Range for $\frac{\bar{\tau}}{(\gamma - \gamma_1) d}$ is 0.14 to 3.0 for the regimes of $\overline{C}$ = 1 p.p.m. to 30,000 p.p.m. This suggests that problems of form roughness and silt intensity are similar. The upper limit of field data falls on the firm line AA which is recommended for design. Its equation is $\bar{\tau} / (\gamma - \gamma_1) d = 0.05 + C^{1/3}$

**Introducing tractive force in terms of q & S*, we have.**

**Fifth Relation.**

$$\bar{C} = \phi\ (q^{2/3} S^*) \qquad \text{Fig. 5 (e)}$$

where q = discharge intensity in Cs. and S* = Slope per thousand.

Individually the trend is very regular for each range of different diameters.

Introducing fall velocity. W, in equation 5 (c), we have.

**Sixth Relation.**

$$\bar{C} = \phi\ (q^{2/3}\ S^*/W) \qquad \text{Fig. 5 (f)}$$

Remarks are same as for relation (5) above.
Introducing square root of fall velocity; we have,

**Seventh Relation.**

$$C = \phi\ \left(q^{2/3}\ S^*/W^{1/2}\right) \qquad \text{Fig. 5 (g)}$$

It will be noted that in this plot, the data of all the tests series falls around a single curve. It appears that this curve is a representative one as it covers a very wide range of silt intensities ranging from 1 p.p.m. to 40,000 p.p.m. (or .001 to 40 gr/litre). The equation of the curve is :—

$$q^{2/3}\ S^*/W^{1/2} = 0.5 + 5\ C^{2/3} \qquad \ldots\ldots\ (21)$$

$\bar{C}$ = gr. per litre.

q = discharge intensity in cusecs.

S* = Slope per thousand.

W = fall velocity of mean size of bed material.

It may also be noted that the above form is similar to Mayer-Peters (13) bed load formula :—

$$q^{2/3}\ \ S/d = a + b\left(\frac{qs}{d}\right)^{2/3} \qquad \ldots\ldots\ (22)$$

where qs = silt as bed load in kg./m. sec.
d = dia of bed material in metres.

The derived equation for silt carrying capacity of a channel can be simplified by eliminating the constant 0,5 when,

$$q^{2/3}\ S^*/W^{1/2} > 5$$

the equation reduces to :—

$$q^{2/3} S^*/W^{\frac{1}{2}} = 5\ C^{2/3}$$

or $$\bar{C} = 0.1\ q.\ S^{2/3}/W^{3/4} \quad \ldots\ldots (23)$$

The general function can be expressed as :

$$\bar{C} = \phi\ (Re, S, W) \quad \ldots\ldots (24)$$

where Re=Reynolds No. $q/\gamma$

Where $\gamma$ is kinematic viscosity, W=fall velocity of bed material

According to dimensional analysis the modified non-dimensional parameters assume the form :

$$\bar{C} = \phi\ (S, Re, F, d/D, W/V,) = O \quad \ldots\ldots (25)$$

where F is Froude No. and other variables have the meaning previously mentioned.

**VI. Derived conclusions from the equation**

$$\bar{C} = O.1\ q.S^{3/2}/W^{3/4}$$

(*i*) For two channels with the same bed silt size their silt carrying capacities are :

$$C_1/C_2 = q_1/q_2 \times (S_1/S_2)^{3/2}$$

(*ii*) For channels with the same value of $S/W^{1/2}$

$$C_1/C_2 = q_1/q_2$$

Silt carrying capacities are in the ratio of their discharge intensities.

(*iii*) Also when $C_1/C_2$ and $W_1/W_2$ are both=1

$$S_1/S_2 = \left[q_2/q_1\right]^{2/3}$$

Compare it with Lacey's slope relation $S=0.39\ f^{5/3}/q^{1/3}$ with the same f for two channels

$$S_1/S_2 = \left[q_2/q_1\right]^{1/3}$$

(*iv*) (a) For a river distorted model to scales horizontal : vertical 1/200, 1/40, distortion 5/1, silt dia O.4 m/m in river but on model O.22 m/m

$$\frac{C_1 \text{ (model)}}{C_2 \text{ (river)}} = \frac{q_1 S_1^{3/2}/W_1^{3/4}}{q_2 S_2^{3/2}/W_2^{3/4}}$$

$$= \frac{(5)^{3/2}}{(40)^{3/2}} \times 1.60$$

$$\frac{C_{(model)}}{C_{(river)}} = 7\%$$

It yields a general equation for scale ratio of silt intensities in model to prototype as :

$$C_{model}/C_{river} = (\text{Distortion} \times \text{vertical scale})^{3/2}/m_r^{3/4} \quad (26)$$

$$= D_r^3 /L_r^{3/2}.m_r^{3/4}$$

where $L_r$, $D_r$ = horizontal & vertical scales

$m_r$ = ratio of fall velocity w of bed silt size in river and model.

(*b*) For a small scale model, let

$L_r$, $D_r$ = 1/1000, 1/100 and $m_r^{3/4} = 1/1.6$

$$C_{model}/C_{river} = (10/100)^{3/2}/m_r^{3/4} = 1.6/31.5$$

$$C_{model}/C_{river} = 5\%$$

The two models with different distorted scales may be expected to yield similar results if scale ratio of silt intensities $C_{model}/C_{river}$ for both the model is the same.

**VII. Practical applications of the silt intensity function for design of alluvial channels.**

$$q^{2/3} \cdot S^{1/2}/W^{2/3} = 0.5 + 5\,C \quad \ldots\ldots (27)$$

CASE—(*i*) When there is no choice of slope.

(*a*) The formula suggests that the most important part is the estimation of silt size of the bed of the channel which will ultimately result from river flow passed into the channel. This can be estimated if we apply the function :

$$m_1/m_2 \propto \phi\,(q_1/q_2)$$

According to Inglis,[3] Indo-Pakistan data shows that

$$m \text{ or } W \propto Q^{1/10} \quad \ldots\ldots (28)$$

but according to Professor C.M. White's[3] paper of 1939 on "The influence of transported solids upon rivers".

$$m \propto Q^{1/5} \quad \ldots\ldots (29)$$

Before the coming in of link canals the ratio of canal/river discharge was very much less but now it will increase by 2 to 3 times the previous values.

Let us take Trimmu Sidhnai Link.

Existing Haveli Q = 5,000 Cs.

Haveli + Link Q = 5,000 + 11,000=16,000 Cs.

Ratio of discharges in pocket = 16,000/5,000=3.2

Let existing maximum silt size entering the pocket and the canal be = 0.25 m/m

Silt size entering the pocket according to Prof. Whites, equation when coarser material is present in the river, will be = 0.25 × 1.265 = 0.315 m/m

Lacey's silt factor f = $1.76\sqrt{m}$ = 0.99

say . . 1.0

In the design, f has been adopted as 0.89, which appears to be on the low side. The most appropriate value is f=1. However f=0.89 can be achieved and excessive silt entry into existing Haveli or Link reduced if silt excluders are used at the Headworks.

(*b*) **Silt carrying capacities of proposed Link canals.**

The silt carrying capacities of all the proposed link channels have been calculated in table IV. The permissible amounts of silt ranges 0.4 to 0.6 gr/litre. Anything in excess than the permissible value is objectionable and should be excluded for maintaining the stability of such channels.

CASE—(*ii*) **If choice of slope is available.**

The formula of sediment concentration :

$$q^{2/3} \cdot S^*/W = 0.5 + 5\, C^{2/3} \quad \ldots\ldots (21)$$

suggests that D and V being variables depending upon the silt transport, they may not be as important as q and s, the discharge intensity and the

slope. Knowing W, for a given q and c, the slope can be determined from the silt transport function.

**VIII. Solved examples for designing a channel.**

**Example I.**

To design a seepage drain to carry no silt for Q=150 Cs.

Adopting Laceys' Q/B ratio = $q = 0.4Q^{1/2}$ = 5 cusecs.

Silt sixe m=0.32 m/m.

Assume silt intensity C=.001 gr. per litre.

Applying $q^{2/3} S^*/ W^{\frac{1}{2}} = 0.5 + 5 (C)^{2/3}$

$$\frac{2.95}{0.37} S^* = 05$$

Slope per thousand=.063

$$S = 1/16000$$

Using shear stress function in fig. 5 (d)

against $\bar{C}$ read $= \phi\left[\frac{\bar{\tau}}{(\gamma - \gamma_1) d}\right]$

Adopting line AA for design $\phi(\bar{\tau}) = 0.145$

$$= 87 \times \frac{.145}{1000} = 0.0124$$

Corelating D from $\bar{\tau} = \gamma DS$

Depth $D = .0125 \times \frac{16000}{62.5}$

D=3.33 ft.

V=5/3.3 = 1.5 ft/sec.

Lacey $f = 0.75 V^2 / D = 0.52$

F=0.148

S=1/16,000

**Example II.**

To design an Irrigation channel :

Q = 16,000 Cs.

$q = 0.4 Q^{\frac{1}{2}}$

q=50 cusecs

Assume $\bar{C}$ = 0.6 gr/litre

m=0.33 m/m

$q^{2/3} .S^*/W^{\frac{1}{2}} = 0.5 + 5 (0.6)^{2/3}$

$$\frac{13.5}{0.375} \times S^* = 4.05$$

Slope per thousand = 0.108

Against $\overline{C}$ = 0.6 gr/litre

read value $\phi\left[\frac{\bar{I}}{(\gamma - \gamma_1)\, d.}\right]$ from fig. 5 (d) taking line AA

It reads 0.93

Substituting the values we have

$$\phi(\bar{I}) = \frac{62.5 \times D \times 1000}{87 \times 9400} = 0.93$$

gives :—

D = 12.3 ft.

V = q/D = 4.05 ft/sec.

S = 1/9200

Lacey f = 1.0

$F = V/\sqrt{gD} = 0.2$

**Example III.**

Let Q = 16,000 Cs.

$q = 0.4\ Q^{\frac{1}{2}}$

q = 50 cusecs

C = 1 gr/litre

m = 0.33 m/m

Reading from the curve 5(g) or from the application of formula (21)

$$\frac{13.5 \times S}{(0.14)^{\frac{1}{2}}} = 0.5 + 5\,(C)^{2/3}$$

$S^* = 5.5/37.5 = 0.15$ per thousand

Using curve at fig. 5(d) against

C = 1 gr/litre ;

the function $\phi(\bar{I}) = 1.1$

solving by substituting values in $\phi(\bar{I})$

$$\frac{62.5\ D.S}{(87)\ d} = 1.1$$

$$= \frac{61.5 \times D \times 0.15 \times 1000}{87 \times 1000}$$

gives :—

D = 10.3 ft.

$V = 50/10.3 = 4.85$ ft/sec.

$S = 0.15$ per thousand.

Lacey $f = 1.72$

Froude No. $= V/\sqrt{gD} = 0.27$

**Example IV.**

Designing Ejector Escapes (with pitched banks) ;

Ejector constructed in canals may be required to work on a maximum of 10 % of canal discharge.

Let escape discharge $Q = 1000$ cusecs

Adopt a narrow section $q = Q^{\frac{1}{2}}$

Say $q = 30$ cusecs.

Silt size $m = 0.4$ m/m

Let main canal discharge = 10,000 cusecs.

and silt concentration $C_1 = 1.0$ gr/litre

Permissible silt intensity $C_o = 0.5$ gr/litre

Silt concentration in escape channel $C_2 = 5$ gr/litre

Apply formula

$$q^{2/3} \cdot S^*/W^{1/2} = 0.5 + 5\,(C2)^{2/3}$$

$$9.7 \times \frac{S}{(0.16)^{1/2}} = 0.5 + 14.5 = 15$$

$S^* = 15/24.3 = 0.62$

Using curve at fig. 5 (d)

$C = \phi(\bar{I})$

against $C = 5$ gr/litre function

$\phi(\bar{I}) = 1.8$

Solving the equation

$$\frac{61.5 \times D \times S \times 305}{87 \times 0.4} = 1.8$$

$D = 5.3$ ft.

therefore $V = 30/5.3 = 5.62$ ft/sec.

$$F = V/\sqrt{gD} = 5.65/\sqrt{32 \times 5.3} = 0.44$$

V & D can also be calculated by applying Mannings' equation, if a suitable $n$ is adopted. As friction reduces at high sediment transport as a result of flattening of bed dunes or waves, adopt $n = .017$.

$$q=\frac{1.49}{0.017}R.^{5/3}.S^{1/2}$$

$$30=93(R)^{5/3}(0.62/1000)^{1/2}$$

$$R^{5/3}=13.1$$

$$R=4.7 \text{ ft.}$$

Velocity can be evaluated by the channel section.

## SUMMARY.

Two functional relations for silt transport in suspension, viz :

(*i*) $q^{2/3}.S^{*}/W^{1/2}=0.5+5\,(C)^{2/3}$

(*ii*) $(\bar{\tau})\,/\,(\gamma-\gamma_1).\,d=.05+C^{1/3}$

are presented in this paper. These functions are considered to be applicable to all such channels where the bed is alluvial and all bed particles are free to change the bed pattern or form roughness as influenced by the flow characteristics :—

Utilising Lacey's equation

$$P=2.67Q^{1/2}$$

$$q=0.4Q^{1/2}$$

Seepage drains to carry very low silt charge, or channels, carrying normal or a little more charge can be designed. Even ejector escape channels to carry super silt charge can be designed from the proposed relations in a rational way.

It is thus now possible to design a channel in relation to its silt carrying capacity. The functions cover a fairly wide range from a lower regime of plane bed when there is almost silt clear to a higher regmine of flow in which case 10 to 40 gramms per litre of silt is carried by the flow.

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# TABLES

**TABLE**

**SUM**

| Stations | Q Discharge | m | V in ft./sec. | D in ft. | S* % o | $\sqrt{gDS}/W$ |
|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| 1. Jhang Branch R. D. 7,260 | 2941·98 | ·14 | 2·93 | 7·43 | ·238 | 1·34 |
| 2. Rakh Branch R. D. 7,260 | 1146·33 | ·43 | 2·58 | 5·35 | ·190 | 1.06 |
| 2-A Rakh Branch R. D. 2,35,620 | 404·77 | ·25 | 2·14 | 3·00 | ·169 | 1·25 |
| 3. Udeki Disty : R. D. 2,200 | 43·46 | ·23 | 1·50 | 1·72 | ·300 | ·73 |
| 4. Main Line Lower R. D. 1,49,000 | 4687·70 | ·45 | 2·98 | 8·41 | ·199 | 1·28 |
| 7. Hafizabad Disty : R. D. 3,350 | 20·86 | ·27 | 1·33 | 1·81 | ·275 | 1·19 |
| 8. Rakh I R. D. 2,13,880 | 582·72 | ·26 | 2·27 | 4·47 | ·144 | 1·34 |
| 10. Burala Branch R. D. 6,000 | 1579·30 | ·34 | 2·65 | 6·34 | ·217 | 1·52 |
| 13-A Lower Gogera R. D. 2.59,000 | 1124·47 | ·26 | 2·39 | 5·68 | ·184 | 1·70 |
| 13-B Lower Gogera R. D. 2,72,500 | 847·21 | ·25 | 2·36 | 5·58 | ·185 | 1·80 |
| 13-C Lower Gogera R. D. 11,000 | 1857·19 | ·33 | 2·65 | 6·16 | ·211 | 1·48 |
| 14-A Tarkhni Disty : R. D. 19,500 | 201·67 | ·23 | 1·99 | 2·58 | ·216 | 1·43 |
| 14-B Lukmania Disty : R. D. 10,000 | 105·39 | ·23 | 1·68 | 2·27 | ·232 | 1·38 |
| 14-C Khurrianwala R. D. 5,000 | 293·33 | ·28 | 2·11 | 2·99 | ·223 | 1·27 |

I—(condt.)

MER

| $F=V/\sqrt{gD}$ | $V^3/D$ | $8gV^2 S$ | $q^{2/3}/S^*/W^{\frac{1}{2}}$ | $\frac{\tau}{(\gamma-\gamma_1)d}$ | Silt intensity in gr/litre Average for 1940, 1942 & 1943 | | |
|---|---|---|---|---|---|---|---|
| | | | | | C | M | (C + M) |
| 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| ·19 | 3.39 | ·520 | 4·319 | ·89 | ·062 | ·284 | ·346 |
| ·19 | 3·21 | ·324 | 2·619 | ·52 | ·151 | ·179 | ·330 |
| ·21 | 3·27 | ·20 | 1·843 | ·45 | ·059 | ·214 | ·273 |
| ·70 | 1·96 | ·173 | 1·78 | ·49 | ·026 | ·311 | ·337 |
| ·17 | 3·14 | ·454 | 4 | ·84 | ·062 | ·232 | ·294 |
| ·17 | 1·39 | ·56 | 1·49 | ·41 | ·050 | ·116 | ·216 |
| ·18 | 2·62 | ·190 | 2·103 | ·54 | ·065 | ·282 | ·350 |
| ·18 | 2·94 | ·390 | 3·61 | ·88 | ·135 | ·322 | ·457 |
| ·17 | 2·40 | ·264 | 3·206 | ·88 | ·110 | ·262 | ·372 |
| ·17 | 2·36 | ·262 | 3·226 | ·90 | ·273 | ·222 | ·495 |
| ·18 | 3·02 | ·38 | 3·708 | ·86 | ·140 | ·332 | ·472 |
| ·21 | 3·05 | ·219 | 2·05 | ·53 | ·071 | ·305 | ·376 |
| ·19 | 2·08 | ·167 | 1·81 | ·57 | ·069 | ·271 | ·340 |
| ·21 | 2·74 | ·260 | 2·717 | ·48 | ·041 | ·658 | ·699 |

**TABLE I**

WIN

| Stations | Q Discharge | m | V in ft./sec. | D in ft. | S* % o | $\frac{\sqrt{gDS}}{W}$ |
|---|---|---|---|---|---|---|
| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| 1. Jhang Branch R. D. 7,260 | 2679·29 | ·46 | 2·89 | 7·12 | ·222 | 1·23 |
| 2. Bakh Branch R. D. 7,260 | 1076·40 | ·43 | 2·47 | 5·04 | ·204 | ·77 |
| 2-A Rakh Branch R. D. 2,35,620 | 403·12 | ·25 | 2·12 | 3·66 | ·168 | 1·37 |
| 3. Udeki Disty : R. D. 2,200 | 39·39 | ·22 | 1·41 | 1·66 | ·303 | 1·41 |
| 4. Main Line Lower R. D. 1,49,000 | 4452·17 | ·46 | 2·93 | 7·99 | ·250 | ·89 |
| 7. Hafizabad Disty : R. D. 3,050 | 20·04 | ·29 | 1·30 | 1·78 | ·268 | 1·06 |
| 8. Rakh I R. D. 2,13,880 | 563·01 | ·25 | 2·24 | 4·39 | ·143 | 1·37 |
| 9. Mian Ali Branch R. D. 95,000 | 350·71 | ·29 | 1·97 | 3·86 | ·197 | 1·34 |
| 10. Burala Branch R. D. 6,000 | 1283·09 | ·34 | 2·55 | 5·39 | ·196 | 1·34 |
| 11. Shahkot Disty : R. D. 12,000 | 168·86 | ·27 | 1·85 | 2·92 | ·237 | 1·36 |
| 13-A Lower Gogera R. D. 25,9000 | 1094·32 | ·21 | 2·37 | 5·82 | ·206 | 1·79 |
| 13-B Lower Gogera R. D. 2,72,500 | 864·62 | ·26 | 2·34 | 5·13 | ·201 | 1·81 |
| 13-C Lower Gogera R. D. 11.000 | 1867·95 | ·35 | 2·68 | 5·85 | ·212 | 1·40 |
| 14-A Tarkhni Disty : R. D. 19,500 | 204·20 | ·22 | 2·03 | 2·61 | ·211 | 1·46 |
| 14-B Lukmania Disty : R. D. 10,000 | 102·39 | ·23 | 1·61 | 2·28 | ·232 | 1·38 |
| 15-C Khurrianwala R. D. 5,000 | 228·32 | ·28 | 2·07 | 3·04 | ·222 | 1·35 |

TER

| $F=V/\sqrt{gD}$ | $V^3/D$ | $8gV^2$ S | $q^{2/3}/S^*/W^{\frac{1}{2}}$ | $\frac{\tau}{(\gamma-\gamma_1).d}$ | Silt intensity in gr/litre Average 1940, 1942 & 1943 | | |
|---|---|---|---|---|---|---|---|
| | | | | | C | M | + M) |
| 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 |
| ·19 | 3·39 | ·473 | 4·38 | ·758 | ·224 | ·138 | ·362 |
| ·19 | 2·99 | ·318 | 2·64 | ·526 | ·306 | ·112 | ·418 |
| ·19 | 2·60 | ·194 | 2·06 | ·54 | ·062 | ·163 | ·225 |
| ·19 | 1·57 | ·154 | 1·71 | ·375 | ·026 | ·195 | ·221 |
| ·18 | 3·16 | ·550 | 4·83 | ·96 | ·064 | ·167 | ·231 |
| ·17 | 1·24 | ·115 | 1·35 | ·36 | ·009 | .073 | ·082 |
| ·18 | 2·55 | ·184 | 2·09 | ·56 | ·060 | ·158 | ·221 |
| ·17 | 1·98 | ·196 | 2·19 | ·59 | ·041 | ·286 | ·327 |
| ·19 | 3·05 | ·320 | 2·03 | ·68 | ·101 | ·202 | ·303 |
| ·18 | 2·12 | ·197 | 2·27 | ·59 | ·032 | ·262 | ·294 |
| ·17 | 2·29 | ·296 | 3·45 | ·98 | ·174 | ·248 | ·422 |
| ·17 | 2·20 | ·282 | 3·57 | ·98 | ·149 | ·189 | ·348 |
| ·19 | 3·36 | ·512 | 3·2 | ·72 | ·151 | ·226 | ·377 |
| ·22 | 2·26 | ·223 | 2·12 | ·55 | ·066 | ·239 | ·305 |
| ·18 | 1·80 | ·153 | 1·82 | ·505 | ·054 | ·144 | ·198 |
| ·20 | 2.90 | ·242 | 2·26 | ·53 | ·039 | ·283 | ·322 |

## TABLE II
## RIVER RAVI AT SHAHDARA

| Q in cusecs | q | $q^{2/3}$ | Slope | S per $10^3$ | m | W | $S/W^{\frac{1}{2}}$ | $q^{2/3} S/W^{1/2}$ | Silt in gr/litre | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Calculated C | Average Observed C |
| 10,000 | 37·5 | 11·2 | 1/5000 | ·2 | 0·28 | 0·14 | ·535 | 6·0 | 1·2 | 1·2 |
| 20,000 | 52·5 | 14·0 | 1/5000 | ·2 | | | ·535 | 7·5 | 1·7 | 2·1 |
| 30,000 | 65·0 | 16·0 | 1/4000 | ·25 | | | ·67 | 10·7 | 2·9 | 2·6 |
| 40,000 | 75·0 | 17·8 | 1/4000 | ·25 | | | ·67 | 12·0 | 3·8 | 3·2 |
| 50,000 | 84·0 | 19·0 | 1/4000 | ·25 | | | ·67 | 12·8 | 4·2 | 3·6 |
| 60,000 | 92·0 | 20·2 | 1/4000 | ·25 | | | ·67 | 13·6 | 4·5 | 3·9 |
| 80,000 | 106·2 | 22·2 | 1/4000 | ·25 | | | ·67 | 15·0 | 5·4 | 4·4 |
| 100,000 | 118·0 | 24·0 | 1/4000 | ·25 | | | ·67 | 16·1 | 5·7 | 4·9 |

# TABLE III

Table IV. Summary of results for 0.04-mm sand

| Run | $\bar{c}_t$ | $\bar{c}_s$ | V | $y_0$ | S | f | l/n | κ | z | $z_1$ | β |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 101 | 8.34 | 6.90 | 1.87 | 0.566 | 0.00101 | 0.042 | 2.94 | 0.19 | 0.076 | 0.088 | 1.16 |
| 102 | 8.20 | 6.72 | 1.74 | 0.462 | 0.00117 | 0.046 | 3.44 | 0.27 | 0.065 | 0.091 | 1.40 |
| 103 | 5.94 | 5.76 | 1.91 | 0.674 | 0.00086 | 0.057 | 3.22 | 0.24 | 0.082 | 0.088 | 1.42 |
| 104 | 9.70 | 9.48 | 2.13 | 0.478 | 0.00107 | 0.029 | 2.63 | 0.18 | 0.092 | 0.095 | 1.05 |
| 105 | 8.34 | 8.76 | 2.61 | 0.565 | 0.00114 | 0.024 | 5.00 | 0.11 | 0.102 | 0.097 | 0.95 |
| 106 | 3.03 | 3.07 | 1.23 | 0.542 | 0.00081 | 0.075 | 3.12 | 0.25 | 0.050 | 0.098 | 1.98 |
| 107 | 0.75 | 0.75 | 0.85 | 0.380 | 0.00078 | 0.107 | 5.00 | 0.25 | 0.030 | 0.124 | 4.10 |
| 108 | 9.1 | 9.99 | 2.42 | 0.663 | 0.00100 | 0.029 | 4.75 | 0.25 | 0.083 | 0.095 | 1.14 |

Table V. Summary of results for 0.1-mm sand

| Run | $\bar{c}$ | $\bar{v}$ | $y_0$ | S | T | h | l | $2y_0/n$ | f | l/n | κ | z | $z_1$ | β |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.225 | 1.69 | 0.561 | 0.00122 | 20.6 | 0.083 | 0.48 | 13.6 | 0.082 | 3.85 | 0.32 | 0.40 | 0.56 | 1.4 |
| 2 | 0.029 | 1.36 | 0.751 | 0.00055 | 21.8 | 0.077 | 0.48 | 19.8 | 0.058 | 4.35 | 0.33 | 0.32 | 0.72 | 2.25 |
| 3 | 0.014 | 1.33 | 0.928 | 0.00043 | 22.8 | 0.074 | — | 25.0 | 0.058 | 4.75 | 0.41 | 0.45 | 0.72 | 1.60 |
| 4 | 0.270 | 2.31 | 0.947 | 0.00101 | 24.0 | 0.084 | 0.51 | 22.0 | 0.045 | 4.15 | 0.38 | 0.65 | 0.47 | 0.75 |
| 5 | 0.424 | 1.92 | 0.514 | 0.00152 | 21.3 | 0.094 | — | 11.0 | 0.085 | 3.85 | 0.30 | 0.41 | 0.54 | 1.32 |
| 6 | 0.313 | 1.70 | 0.518 | 0.00144 | 21.6 | 0.100 | 0.48 | 10.4 | 0.066 | 3.45 | 0.29 | 0.40 | 0.53 | 1.32 |
| 7 | 0.066 | 1.32 | 0.533 | 0.00106 | 23.3 | 0.108 | — | 9.8 | 0.064 | 3.45 | 0.31 | 0.42 | 0.61 | 1.45 |
| 8 | 0.156 | 1.75 | 0.756 | 0.00092 | 21.6 | 0.100 | 0.56 | 15.2 | 0.060 | 3.45 | 0.31 | 0.42 | 0.55 | 1.31 |
| 9 | 0.061 | 1.59 | 0.995 | 0.00058 | 26.5 | 0.110 | 0.38 | 38.0 | 0.058 | 3.35 | 0.29 | 0.44 | 0.61 | 1.39 |
| 10 | 0.272 | 1.55 | 0.381 | 0.00186 | 24.9 | 0.101 | 0.44 | 7.6 | 0.076 | 3.15 | 0.26 | 0.42 | 0.55 | 1.31 |
| 11 | 0.055 | 1.07 | 0.511 | 0.00160 | 26.4 | 0.109 | 0.44 | 5.8 | 0.112 | 3.55 | 0.36 | 0.49 | 0.65 | 1.33 |
| 12 | 0.103 | 1.28 | 0.382 | 0.00150 | 21.5 | 0.080 | 0.45 | 9.6 | 0.090 | 3.45 | 0.29 | 0.40 | 0.61 | 1.52 |
| 13 | 0.131 | 1.69 | 0.725 | 0.00080 | 23.7 | 0.075 | 0.45 | 19.4 | 0.052 | 4.00 | 0.32 | 0.40 | 0.61 | 1.52 |
| 14 | 0.143 | 1.15 | 0.250 | 0.00210 | 19.8 | 0.063 | 0.45 | 8.0 | 0.102 | 3.25 | 0.31 | 0.34 | 0.63 | 1.85 |
| 15 | 0.515 | 3.38 | 0.472 | 0.00120 | 22.8 | Plane | (2640) | | 0.013 | 4.75 | 0.17 | 0.37 | 0.61 | 0.63 |
| 16 | 0.305 | 2.20 | 0.709 | 0.00107 | 25.2 | 0.072 | — | 19.8 | 0.040 | 3.25 | 0.25 | 0.68 | 0.55 | 0.78 |

ASCE

SEDIMENT LOAD

1550-19

TABLE III (*Continued*)

VARIABLES FOR RUNS WITH 0.78 mm SAND, [illegible] FLUME

| Run (1) | $S_w$ & $10^{-5}$ per ft (2) | Discharge cfs (3) | Depth, in ft (4) | Velocity, in ft per sec (5) | Viscosity x $10^{-5}$ ft$^2$ per sec (6) | Temperature in °C (7) | Total load, in ppm (8) | Bed Forms (9) |
|---|---|---|---|---|---|---|---|---|
| [illegible] | 0.0003 | 3.79 | 0.290 | 0.53 | 1.46 | 9.0 | 0 | Plane |
| [illegible] | 0.0007 | 6.63 | 1.01 | .82 | 1.27 | 13.9 | 0 | Plane |
| 8A | 0.011 | 7.76 | 1.00 | .97 | 1.34 | 11.9 | - | Plane |
| 8B | 0.023 | 7.76 | 1.01 | .96 | 1.38 | 10.9 | 3.3 | Ripple |
| 10 | 0.041 | 4.10 | .59 | .88 | 1.23 | 15.1 | 1.0 | Ripple |
| 5 | 0.045 | 10.73 | 1.00 | 1.34 | 1.19 | 16.5 | 12 | Ripple |
| 12B | 0.052 | 13.45 | 1.00 | 1.68 | 1.19 | 16.4 | 75 | Ripple |
| 4 | 0.069 | 10.73 | 0.86 | 1.56 | 1.25 | 14.6 | 61 | Ripple |
| 11 | 0.073 | 4.92 | 0.59 | 1.04 | 1.23 | 14.9 | 20 | Ripple |
| 39 | 0.090 | 15.74 | 1.06 | 1.86 | 1.15 | 17.6 | 330 | Dune |
| [illegible] | 0.100 | 12.70 | 0.88 | 1.80 | 1.18 | 16.7 | 405 | Dune |
| 12 | 0.108 | 7.19 | 0.57 | 1.58 | 1.20 | 16.9 | 150 | Ripple |
| 14 | 0.116 | 8.01 | 0.62 | 1.74 | 1.21 | 15.6 | 298 | Dune |
| 20 | 0.120 | 18.14 | 1.05 | 2.16 | 1.21 | 15.6 | 506 | Dune |
| [illegible] | 0.131 | 15.19 | 0.92 | 2.06 | 1.21 | 15.8 | 654 | Dune |
| 21 | 0.131 | 20.39 | 1.07 | 2.38 | 1.19 | 16.5 | 752 | Dune |
| 19 | 0.134 | 9.50 | 0.65 | 1.90 | 1.23 | 14.9 | 563 | Dune |
| 16B | 0.134 | 17.23 | 1.03 | 2.11 | 1.21 | 15.8 | 549 | Dune |
| 22 | 0.134 | 22.02 | 0.91 | 3.02 | 1.21 | 15.6 | 1230 | Trans. |
| 17B | 0.136 | 10.01 | 0.65 | 1.92 | 1.24 | 14.7 | 505 | Dune |
| [illegible] | 0.136 | 15.28 | 0.88 | 2.17 | 1.22 | 15.2 | 735 | Dune |
| [illegible] | 0.143 | 11.96 | 0.61 | 2.45 | 1.24 | 14.7 | 1040 | Trans. |
| 30 | 0.142 | 15.68 | 0.64 | 3.06 | 1.25 | 14.5 | 1370 | Trans. |
| 34 | 0.150 | 5.50 | 0.44 | 1.56 | 1.20 | 14.7 | 430 | Dune |
| 32 | 0.153 | 14.92 | 0.60 | 3.11 | 1.31 | 12.7 | 1840 | Plane |
| [illegible] | 0.158 | 12.87 | 0.75 | 2.14 | 1.30 | 13.0 | 789 | Dune |
| 31 | 0.172 | 21.98 | 0.82 | 3.35 | 1.21 | 15.7 | 2350 | Trans. |
| 25 | 0.190 | 21.85 | 0.72 | 3.79 | 1.24 | 14.7 | 2710 | Plane |
| 28 | 0.220 | 15.72 | 0.55 | 3.57 | 1.23 | 15.1 | 2760 | Plane |
| 29 | 0.278 | 15.70 | 0.52 | 3.77 | 1.22 | 15.4 | 3120 | Plane |
| 26 | 0.328 | 15.51 | 0.50 | 3.88 | 1.23 | 15.0 | 5060 | Antidune |
| 35 | 0.470 | 21.76 | 0.58 | 4.69 | 1.38 | 10.8 | 10500 | Antidune |
| 24 | 0.533 | 15.47 | 0.43 | 4.50 | 1.23 | 15.1 | 12300 | Antidune |
| 33 | 0.593 | 21.34 | 0.56 | 4.76 | 1.40 | 10.2 | 13000 | Antidune |
| 27 | 0.815 | 21.33 | 0.54 | 4.93 | 1.38 | 10.9 | 27800 | Antidune |
| 37 | 0.820 | 8.34 | 0.30 | 3.48 | 1.35 | 11.6 | 19900 | Antidune |
| 39 | 0.930 | 15.26 | 0.40 | 4.77 | 1.37 | 11.1 | 36100 | Antidune |
| 36 | 1.667 | 21.38 | 0.57 | 4.69 | 1.36 | 11.5 | 42100 | Antidune |



TABLE I.—BASIC VARIABLES FOR 0.43 mm BED MATERIAL

| Run No. | $S_w$ & $10^{-5}$ ft/ft | Q, cfs | D, ft | V, fps | T, °C | $C_t$, ppm | Form of Bed Roughness |
|---|---|---|---|---|---|---|---|
| 14 | .015 | 3.84 | 0.41 | 0.71 | 16.2 | - - | Plane |
| 17 | .026 | 0.736 | 0.08 | 0.30 | 14.0 | 1 | Ripples |
| 16 | .027 | 5.11 | 0.81 | 0.79 | 14.0 | - | Ripples |
| 15 | .019 | 1.83 | 0.31 | 0.55 | 9.0 | - - | Plane |
| 13 | .033 | 5.07 | 0.80 | 0.79 | 11.0 | 1 | Ripples |
| 18 | .031 | 3.62 | 0.58 | 0.75 | 11.0 | 1 | Ripples |
| 4 | .036 | 7.59 | 0.82 | 1.29 | 11.0 | 94 | Ripples |
| 3 | .035 | 7.59 | 0.65 | 1.35 | 11.5 | 101 | Ripples |
| 9 | .069 | 3.84 | 0.55 | 0.88 | 12.0 | 2 | Ripples |
| 1 | .042 | 7.45 | 0.80 | 1.23 | 9.0 | 23 | Ripples |
| 5 | .047 | 7.93 | 0.75 | 1.32 | 11.0 | 28 | Ripples |
| 11 | .049 | 1.95 | 0.35 | 0.70 | 11.5 | 4 | Ripples |
| 4 | .057 | 7.94 | 0.69 | 1.44 | 10.0 | 91 | Dunes |
| 5 | .060 | 3.83 | 0.51 | 0.93 | 12.0 | 8 | Ripples |
| 7 | .078 | 7.96 | 0.70 | 1.43 | 11.5 | 268 | Dunes |
| 10 | .088 | 1.95 | 0.33 | 0.75 | 10.5 | 20 | Ripples |
| 6 | .088 | 3.90 | 0.46 | 1.07 | 9.5 | 82 | Ripples |
| 12 | .106 | 1.95 | 0.29 | 0.95 | 11.7 | 1 | Ripples |
| 19 | .112 | 4.24 | 0.41 | 1.30 | 18.0 | 208 | Dunes |
| 21 | .114 | 12.12 | 0.96 | 1.58 | 16.0 | 641 | Dunes |
| 22 | .124 | 13.34 | 1.00 | 1.70 | 15.7 | 719 | Dunes |
| 25 | .159 | 8.91 | 0.42 | 1.47 | 17.0 | 378 | Dunes |
| 20 | .103 | 8.14 | 0.61 | 1.68 | 16.4 | 308 | Dunes |
| 23 | .247 | 13.34 | 0.65 | 3.37 | 18.0 | 856 | Dunes |
| 24 | .289 | 8.75 | 0.62 | 1.26 | 17.0 | 1200 | Dunes |
| 40 | .391 | 21.41 | 0.61 | 3.32 | 13.0 | 2460 | Dunes |
| 39 | .364 | 20.94 | 0.55 | 4.71 | 19.0 | 3960 | Transition |
| 26 | .336 | 14.45 | 0.34 | 3.58 | 17.0 | 4580 | Transition |
| 28 | .260 | 11.19 | 0.46 | 3.52 | 16.0 | 4230 | Transition |
| 29 | .369 | 4.54 | 0.30 | 1.89 | 17.4 | 1850 | Transition |
| 31 | .432 | 14.63 | 0.44 | 4.24 | 17.0 | 4750 | Transition |
| 27 | .436 | 7.51 | 0.32 | 2.99 | 15.0 | 4100 | Transition |
| 36 | .446 | 3.15 | 0.18 | 2.04 | 19.0 | 1370 | Transition |
| 41 | .466 | 21.62 | 0.54 | 3.65 | 18.7 | 4340 | Transition |
| 30 | .492 | 5.33 | 0.27 | 2.47 | 17.2 | 3550 | Transition |
| 35 | .494 | 5.39 | 0.28 | 2.80 | 17.0 | 4610 | Transition |
| 34 | .343 | 5.44 | 0.28 | 3.73 | 17.5 | 5690 | Transition |
| 33 | .697 | 10.02 | 0.27 | 4.60 | 16.0 | 6810 | Transition |
| 38 | .819 | 21.35 | 0.30 | 5.53 | 19.0 | 6230 | Transition |
| 37 | .820 | 15.87 | 0.43 | 5.34 | 18.5 | 5570 | Transition |
| 32 | .956 | 14.96 | 0.37 | 5.03 | 18.0 | 6180 | Antidune |
| 45 | .862 | 5.89 | 0.45 | 3.50 | 18.9 | 5630 | Antidune |
| 44 | .898 | 10.82 | 0.28 | 4.78 | 19.4 | 15,100 | Antidune |
| 42 | .986 | 15.43 | 0.34 | 5.76 | 20.0 | 11,200 | Antidune |
| 43 | 1.01 | 21.42 | 0.42 | 5.18 | 18.5 | 11,500 | Antidune |

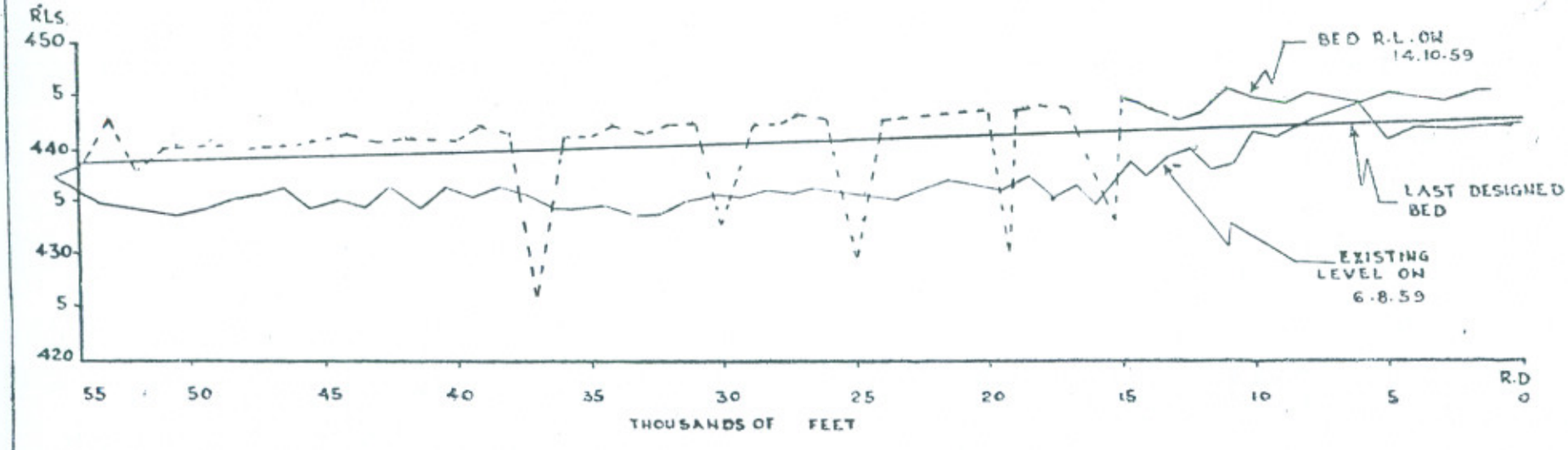
## TABLE IV
## ADOPTED DESIGNED DATA FOR LINKS

| Link | R. D. | m | f | D | V | q | S* | W | $q^{2/3}.S^*/W^{\frac{1}{2}}$ | $\overline{C}$ in gr/litre |
|---|---|---|---|---|---|---|---|---|---|---|
| Trimmu-Sidhnai Q=11,000 Cs. | 0-219 | ·25 | ·89 | 12 | 3·5 | 42 | ·095 | ·1 | 3·6 | 0·5 |
| Sidhnai—Mailsi Bahawal Q=10,100 Cs. | 0-60 | ·25 | ·88 | 11·6 | 3·54 | 41 | ·095 | ·1 | 3·5 | 0·48 |
| Rasul—Qadirabad Q=19,000 Cs. | 0-148 | ·3 | ·97 | 13·8 | 4·03 | 55·7 | 0·1 | ·13 | 4·03 | 0·6 |
| Qadirabad - Balloki Q=18,600 Cs. | 0-88 | ·34 | 1·03 | 13·4 | 4·12 | 55·3 | 0·11 | ·14 | 4·0 | 0·59 |
| L.C.C. Feeder) Q=4.100 Cs. | 0-110 | ·27 | ·91 | 8·6 | 3·09 | 26·6 | 0·107 | ·17 | 3·2 | 0·39 |
| Balloki—Suleimanki II. Q=6,500 Cs. | 72-270 | ·3 | ·99 | 9·8 | 3·4 | 35·3 | 0·13 | ·13 | 3·9 | 0·55 |
| Chashma—Jhelum Q=21,700 Cs. | 0-315 | ·35 | 1·04 | 14 | 21·22 | 59 | 0·11 | ·145 | 4·0 | 0·58 |
| Taunsa—Panjnad Q=12,000 Cs. | 0-188 | ·31 | 0·98 | 11·8 | 3·75 | 44·2 | 0·11 | ·132 | 3·56 | 0·48 |

FIG-1(a)

## L-SECTION OF MAILSI CANAL

CANAL SILTS UP IN 70 DAYS = ——



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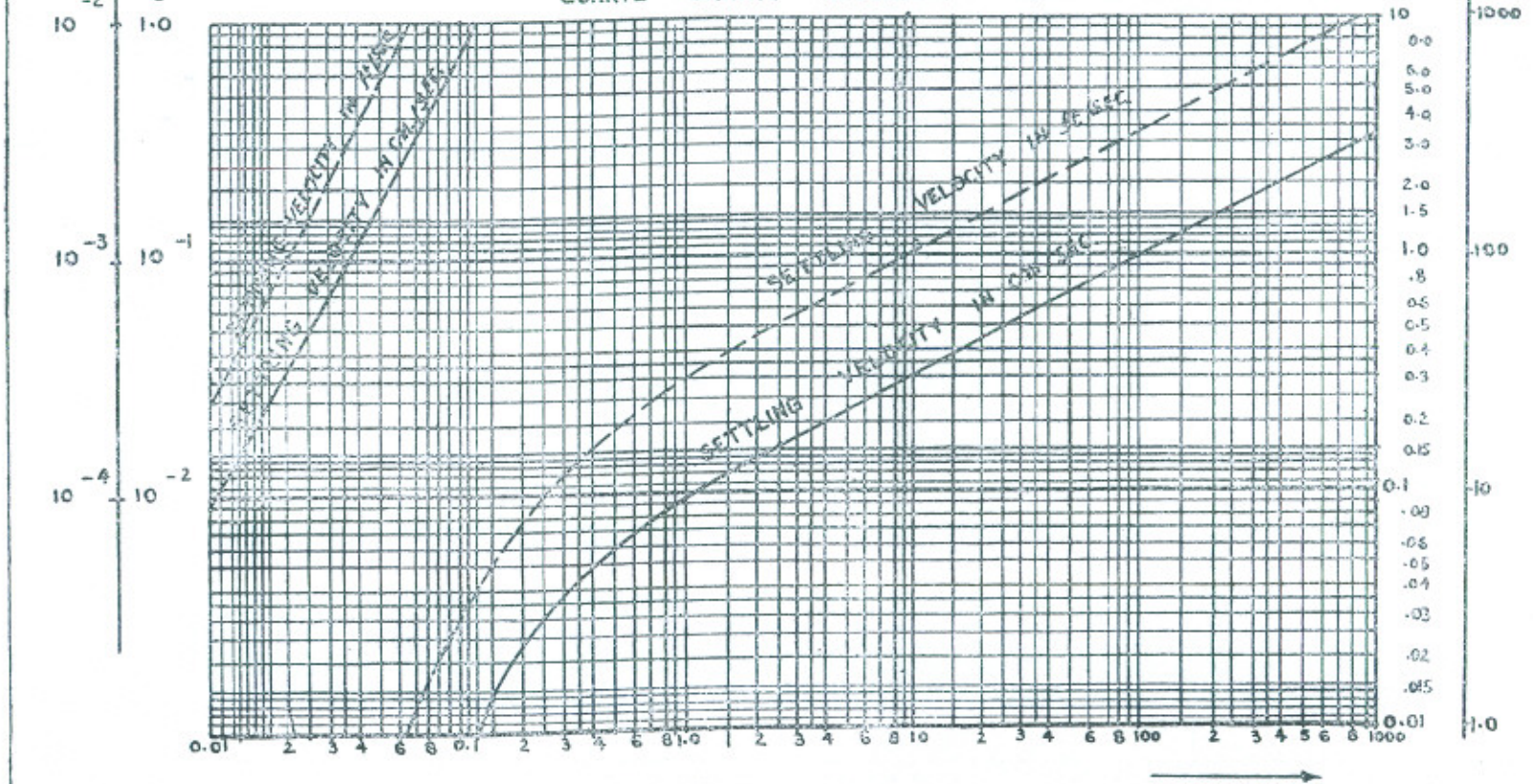
296-60-1

FIG-1(B)

SETTLING VELOCITY Vs FOR VARIOUS SIZE

OF

GUARTZ GRAINS ACCORDING TO RUBBY

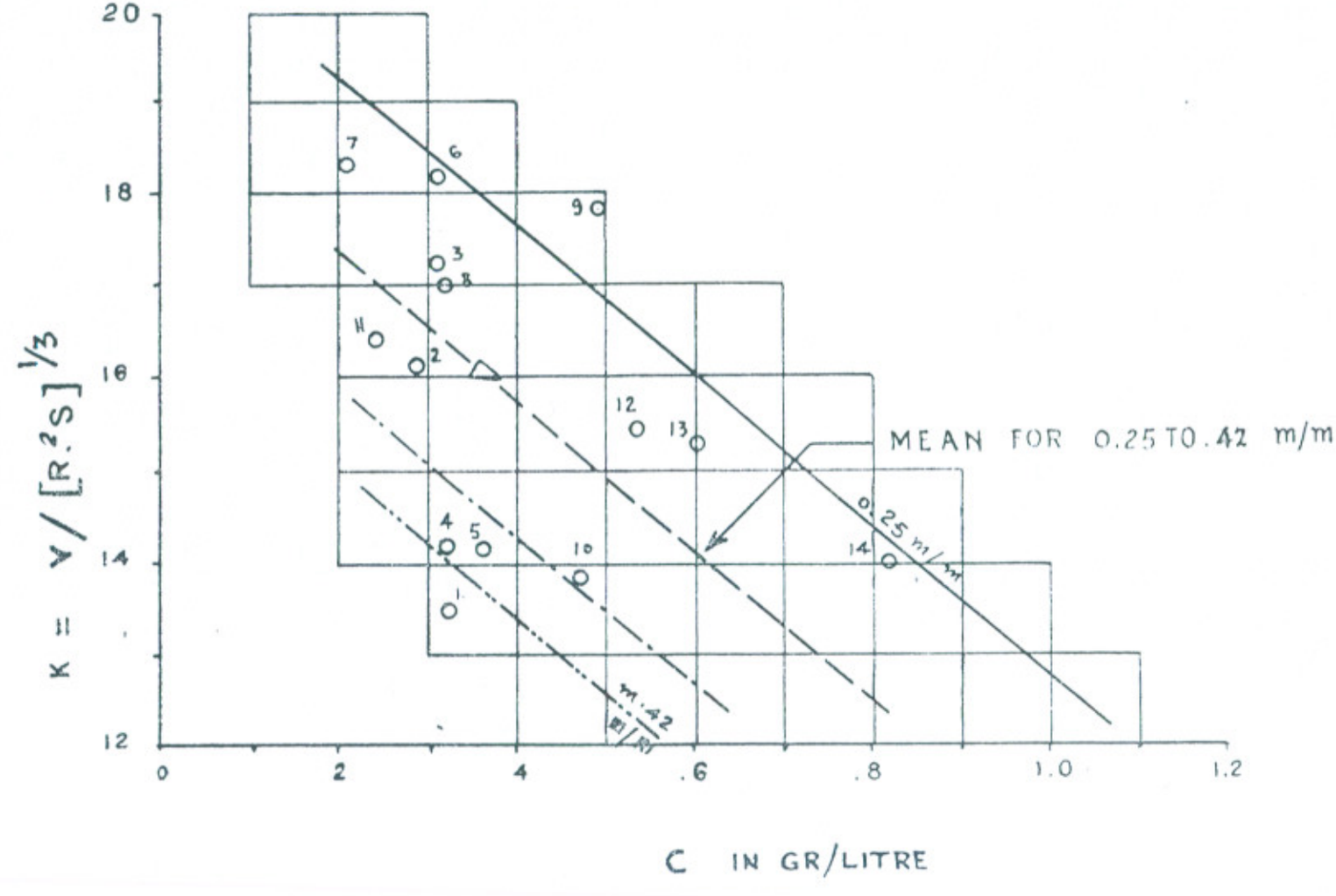



893370.31 (IN POCKET)

T.B.K.M. SALEEM RAZA

FIG : 2(a)

C : $V/(R^2S)^{1/3}$

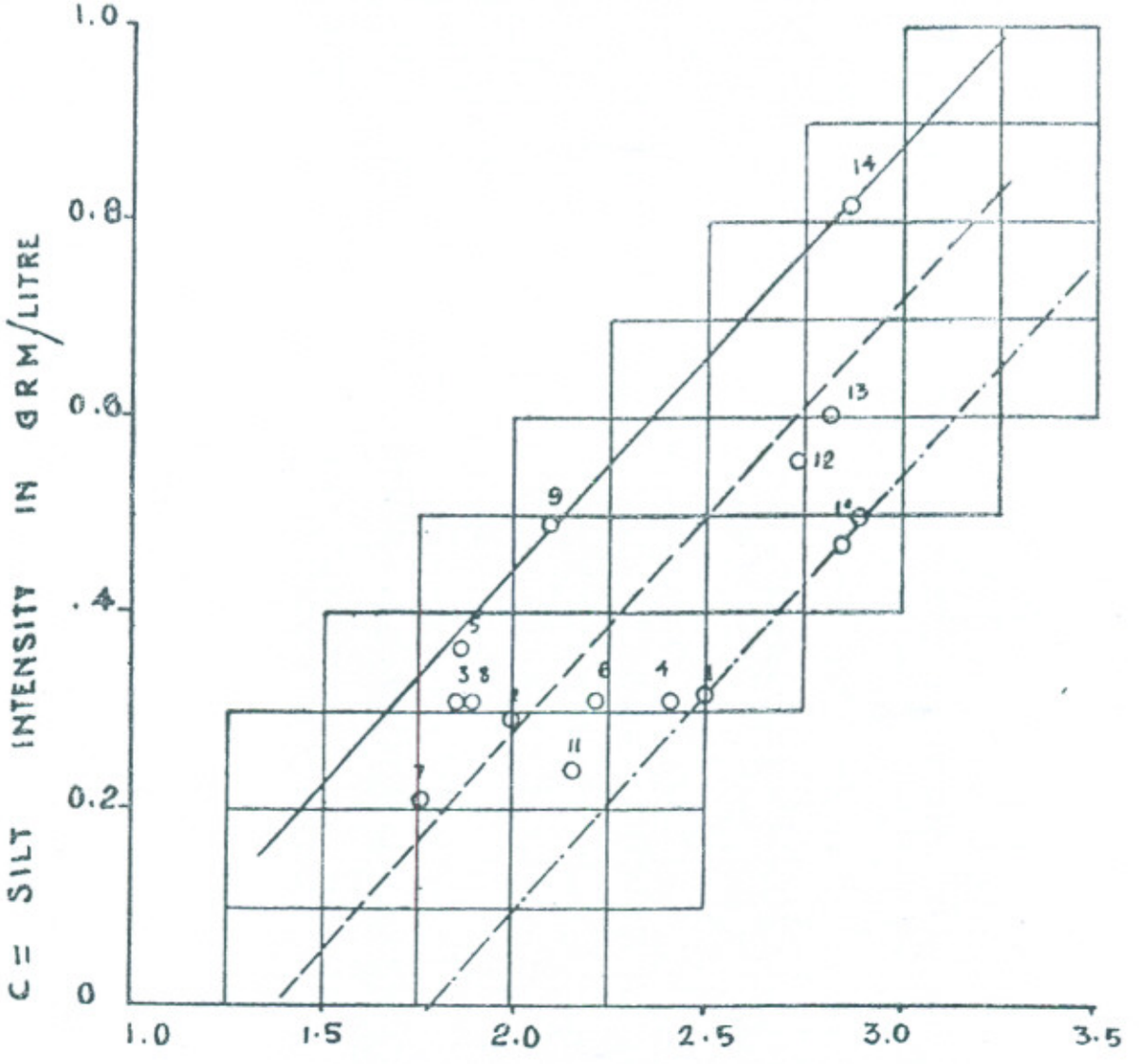


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FIG: 2(2)

$\bar{C} : K_2$

C = SILT INTENSITY
$K_2$ BOSE SLOPE COEF.




$K_2 = S.Q^{.21}/m$ 86

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FIG. 3.

# SILT INTENSITIES OF CHANNELS IN REGIME

COARSE & MEDIUM

$C = \phi_0\left(\frac{V^3}{R}\right)$

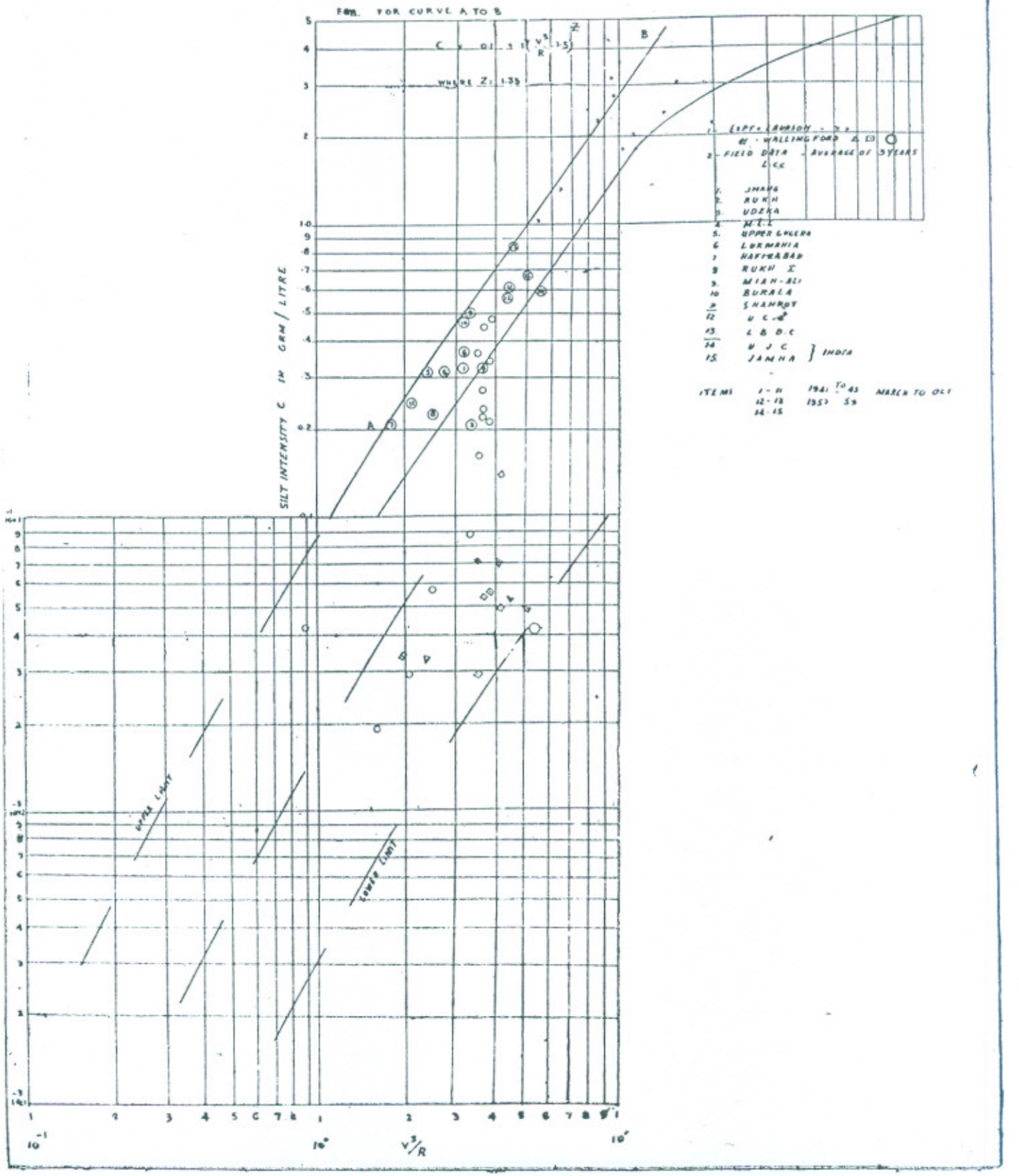
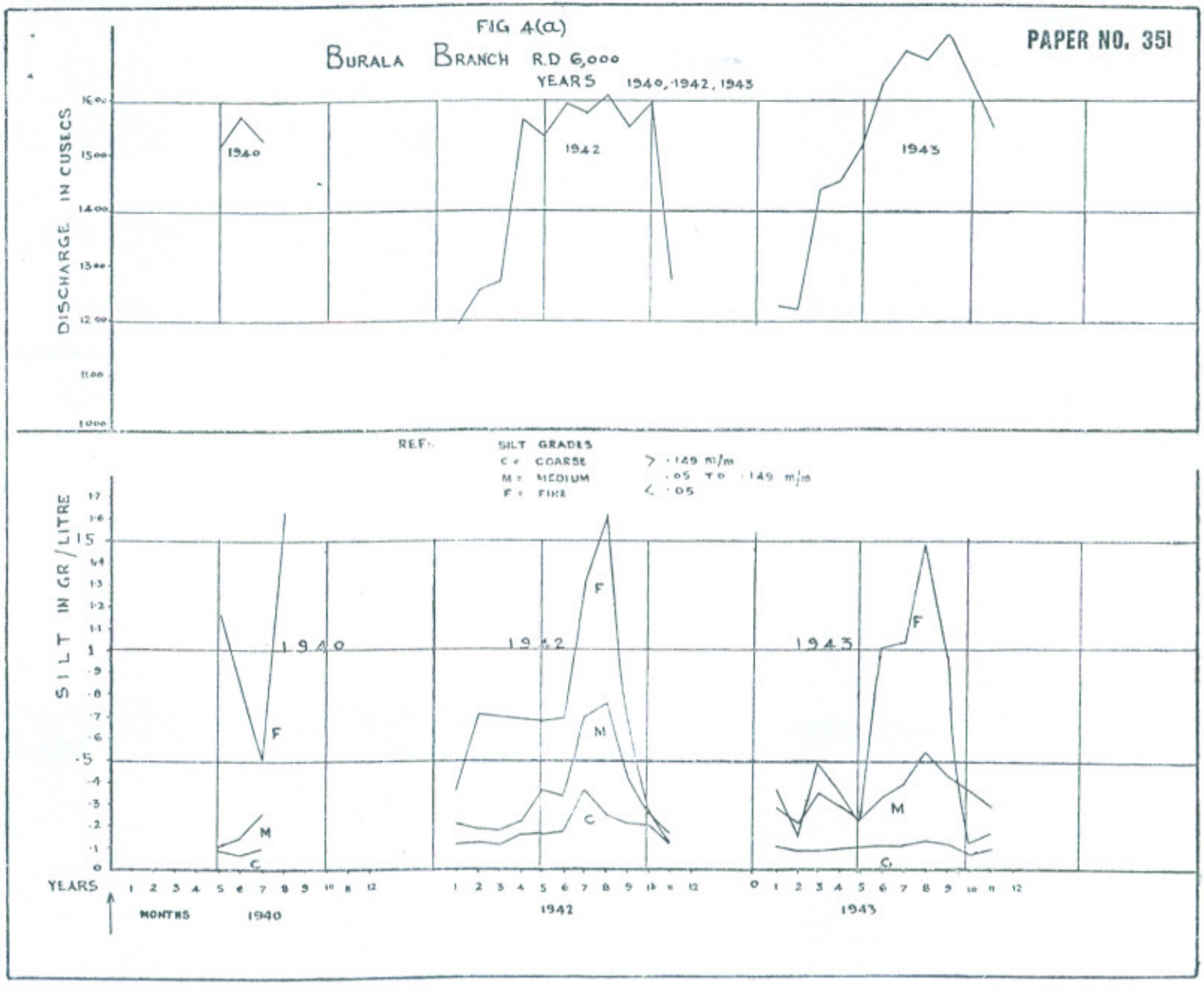


FIG 4(a)

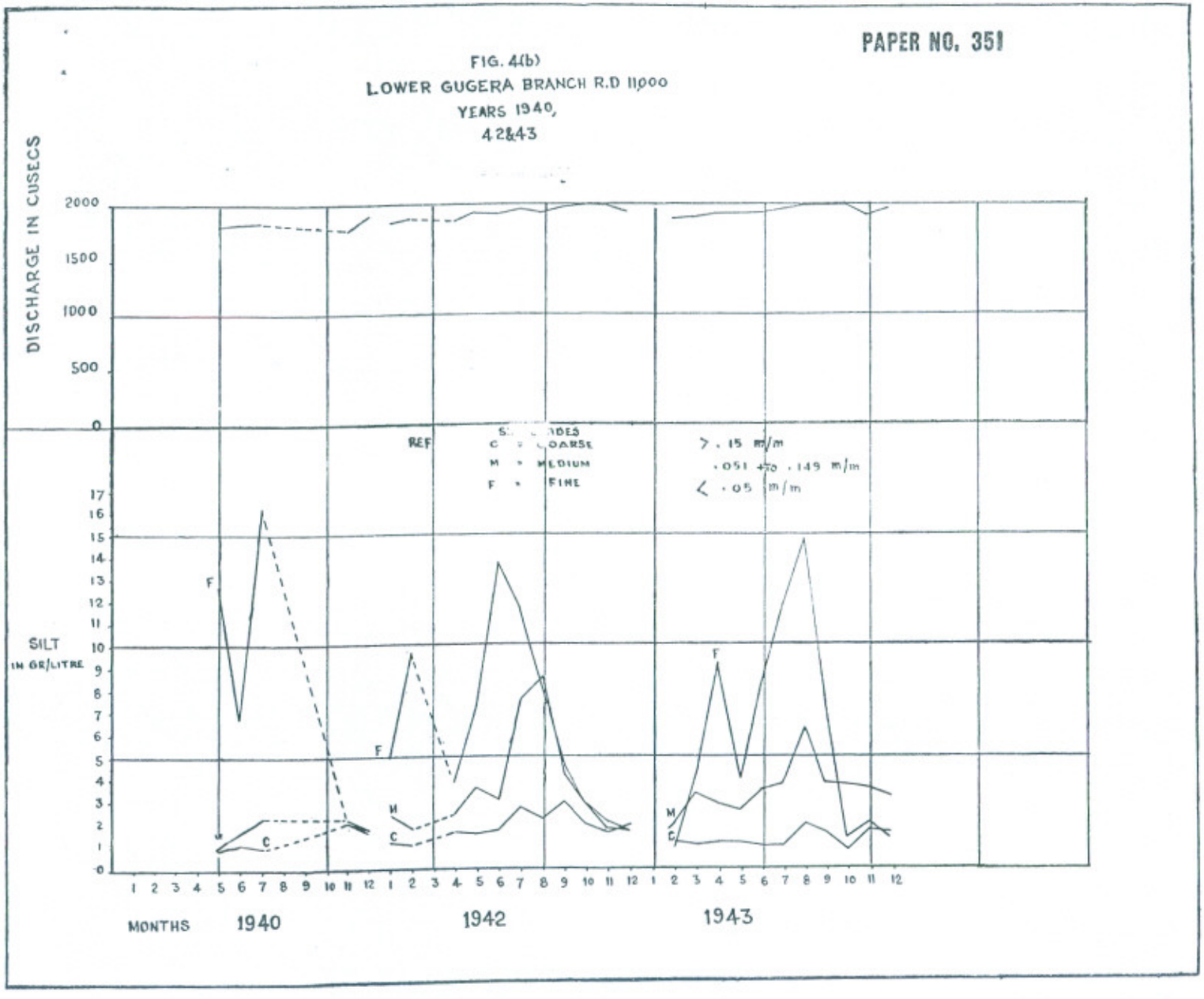
BURALA BRANCH R.D 6,000

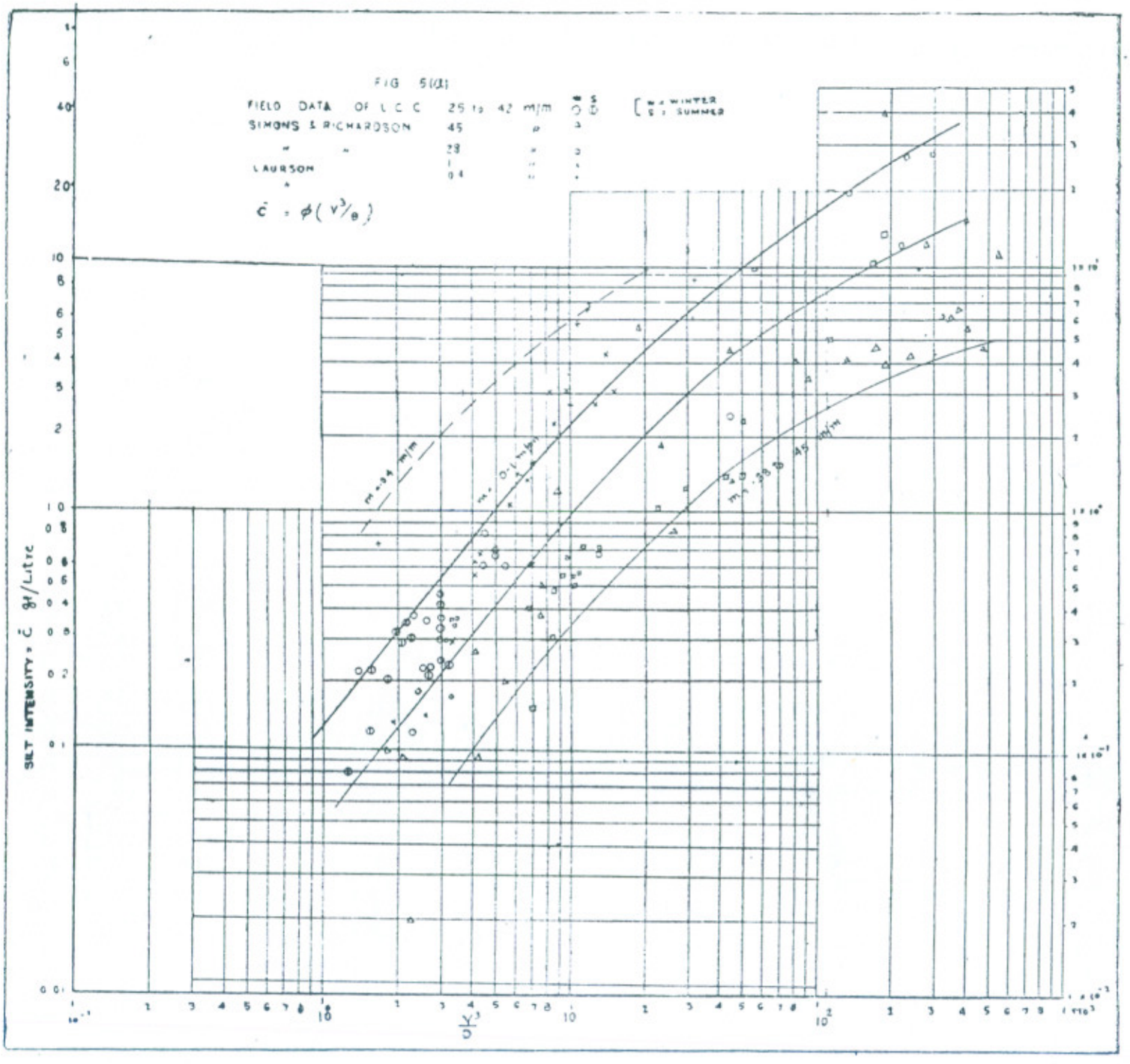
YEARS 1940, 1942, 1943

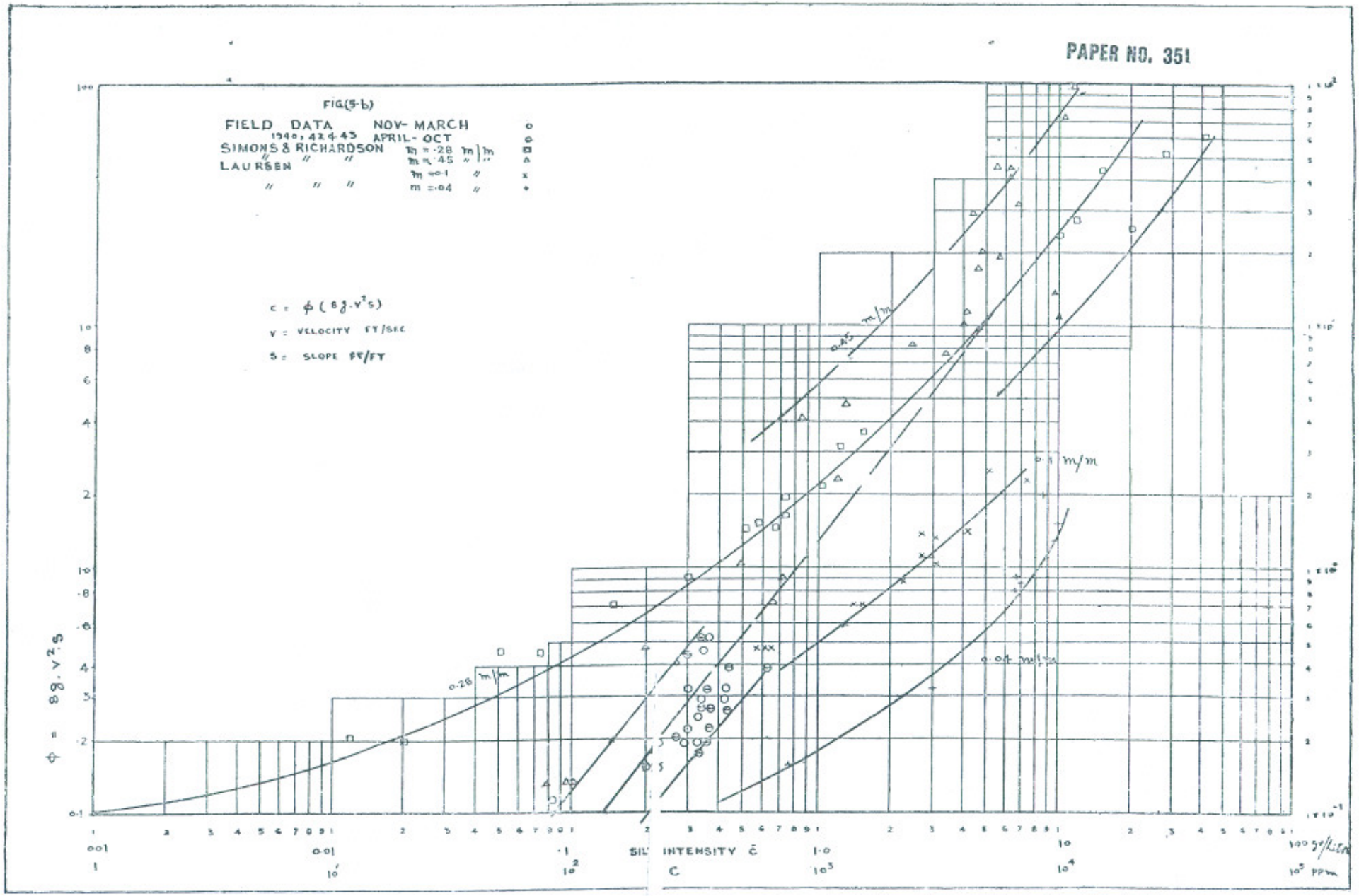


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FIG. 4(b)
LOWER GUGERA BRANCH R.D 11000
YEARS 1940,
42&43




FIG 5(3)
FIELD DATA OF L C C 25 to 42 m/m
SIMONS & RICHARDSON 45
" " 28
LAURSON 1
0.4
$\bar{C} = \phi(V^3/\theta)$
[W = WINTER S = SUMMER
m = 0.4 m/m
m = 0.1 m/m
m = .38 @ 45 m/m
SILT INTENSITY = $\bar{C}$ gm/Litre
$\frac{V^3}{\theta}$


FIG(5-b)
FIELD DATA NOV- MARCH
1940, 42 & 43 APRIL- OCT
SIMONS & RICHARDSON m = ·28 m/m
" " " m = ·45 " "
LAURSEN m = ·1 "
" " " m = ·04 "
c = φ ( δ g · v² s )
v = VELOCITY FT/SEC
s = SLOPE FT/FT
φ = δg · v² s
0·28 m/m
0·45 m/m
0·1 m/m
0·04 m/m
SILT INTENSITY C̄
C
100 gr/litre
10⁵ PPm

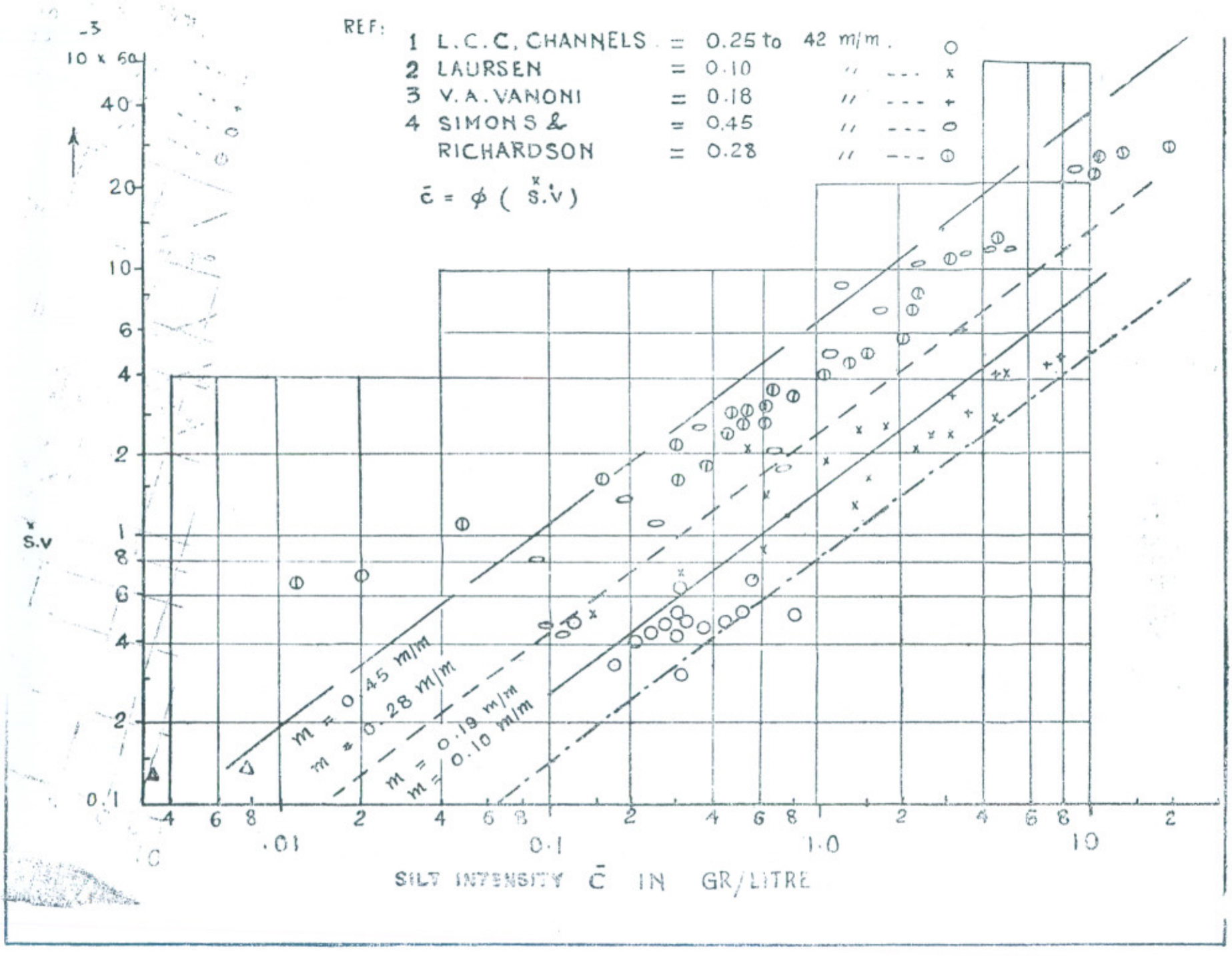
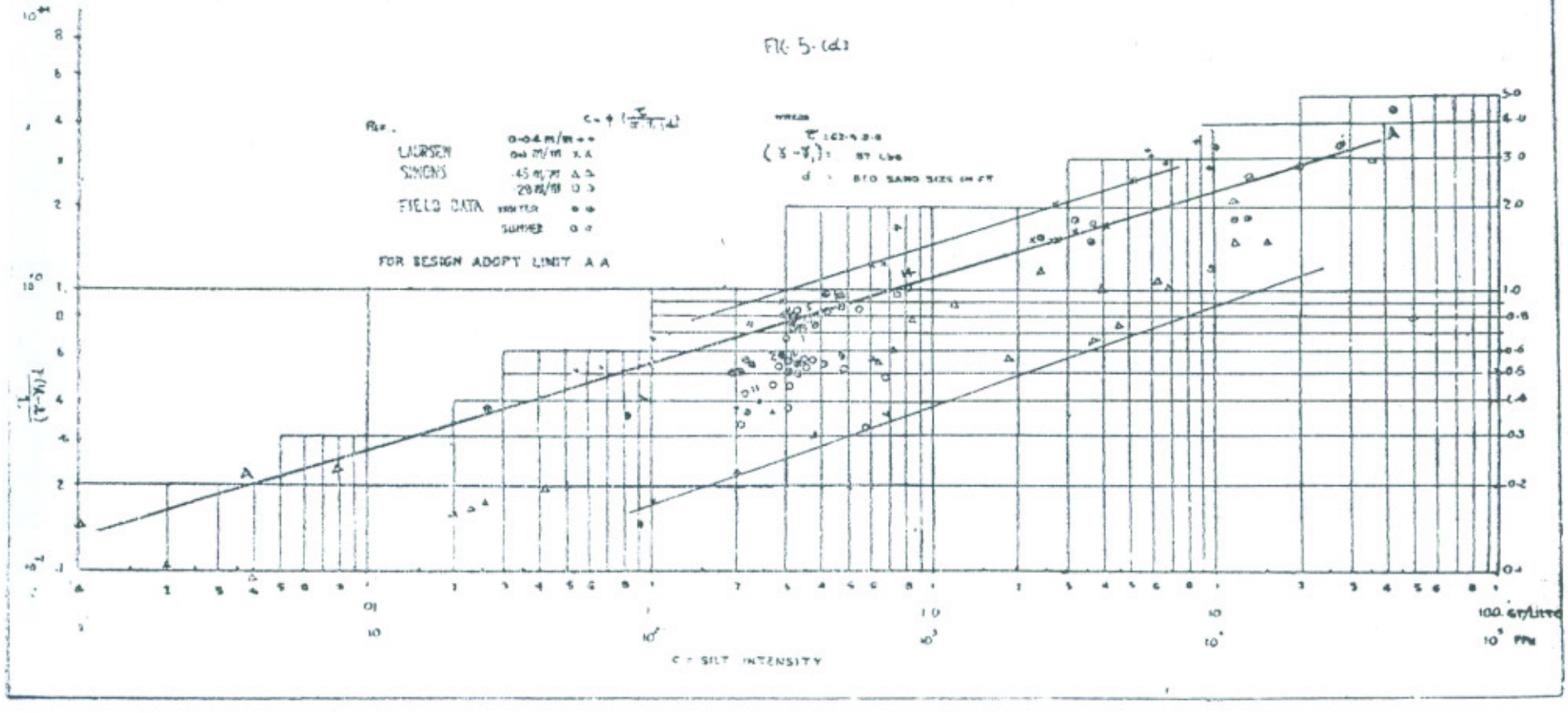

REF:
1 L.C.C. CHANNELS = 0.25 to 42 m/m.
2 LAURSEN = 0.10
3 V.A. VANONI = 0.18
4 SIMONS & RICHARDSON = 0.45
= 0.28
$\bar{c} = \phi\left(\overset{x}{S}.v\right)$
$\overset{x}{S}.v$
m = 0.45 m/m
m = 0.28 m/m
m = 0.19 m/m
m = 0.10 m/m
SILT INTENSITY $\bar{C}$ IN GR/LITRE

FIG 5-(d)


LAURSEN
SINGHS
FIELD DATA
WINTER
SUMMER
FOR DESIGN ADOPT LIMIT A A
C = SILT INTENSITY
100 gm/litre
10³ PPM

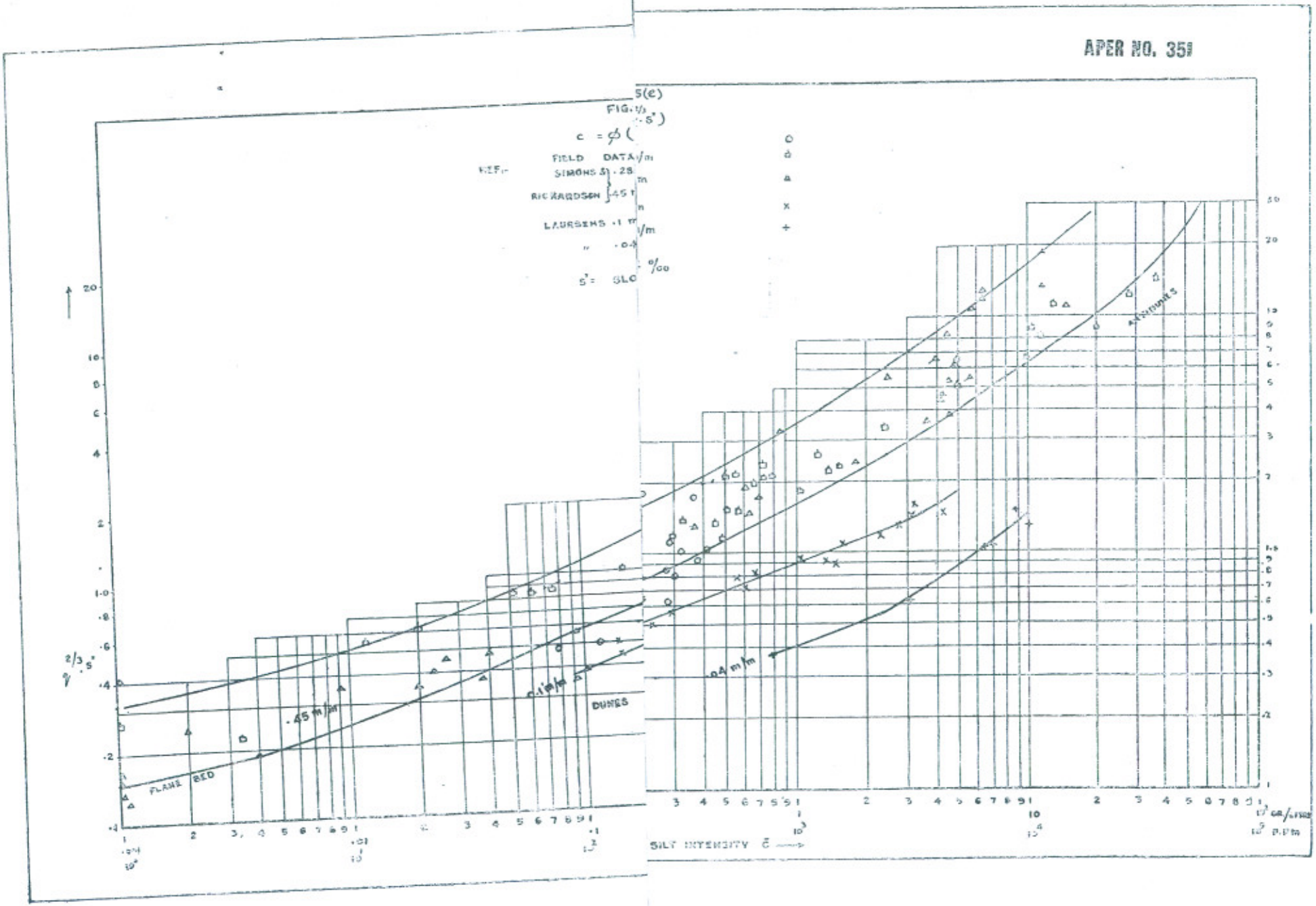
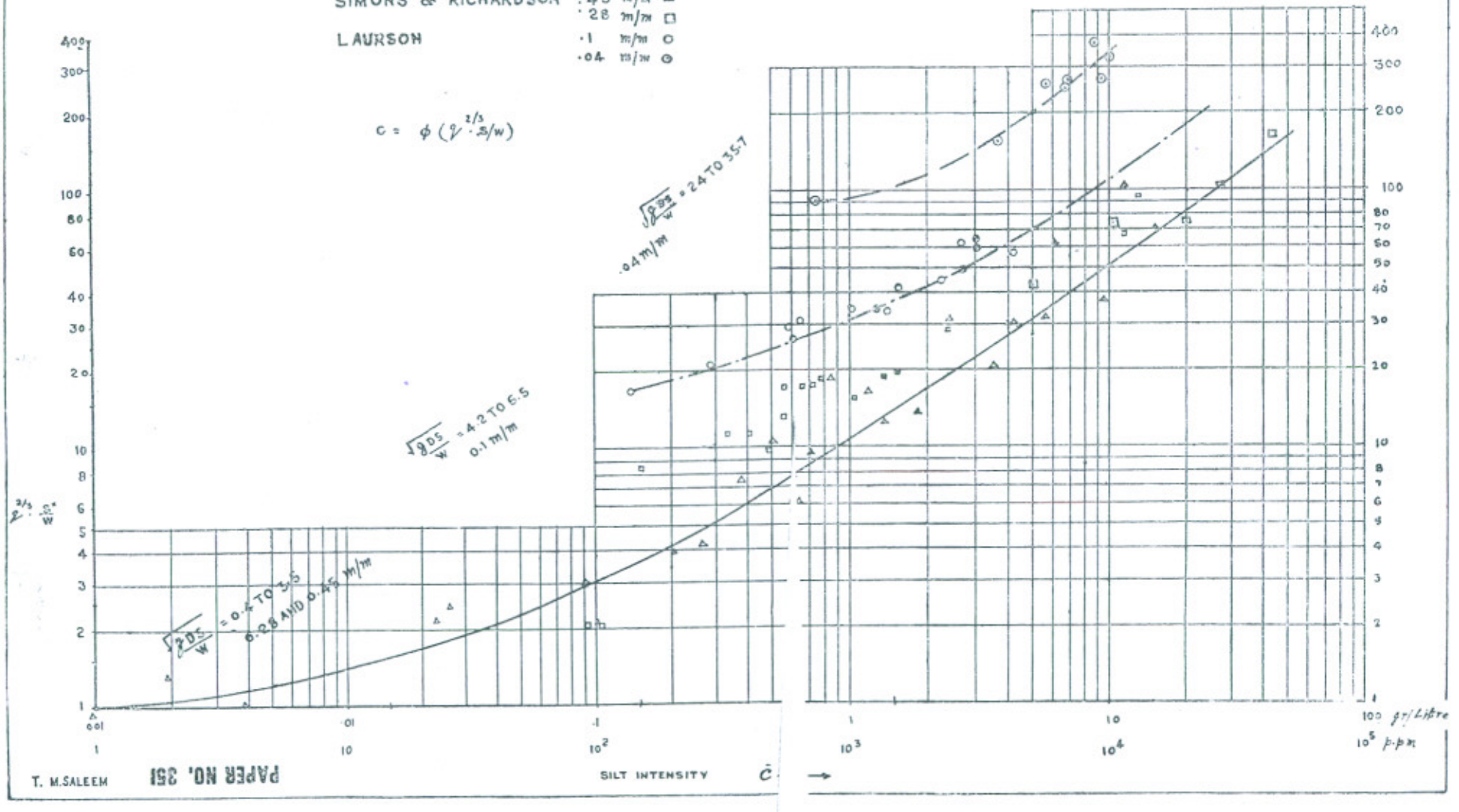

S(e)
FIG.
c = φ(
FIELD DATA
SIMONS
RICHARDSEN
LAURSENS
DUNES
.45 m/m
.1 m/m
.04 m/m
FLUME BED
SILT INTENSITY C

FIG. 5(f)

SIMONS & RICHARDSON .45 m/m △
.28 m/m □

LAURSON .1 m/m ○
.04 m/m ⊙

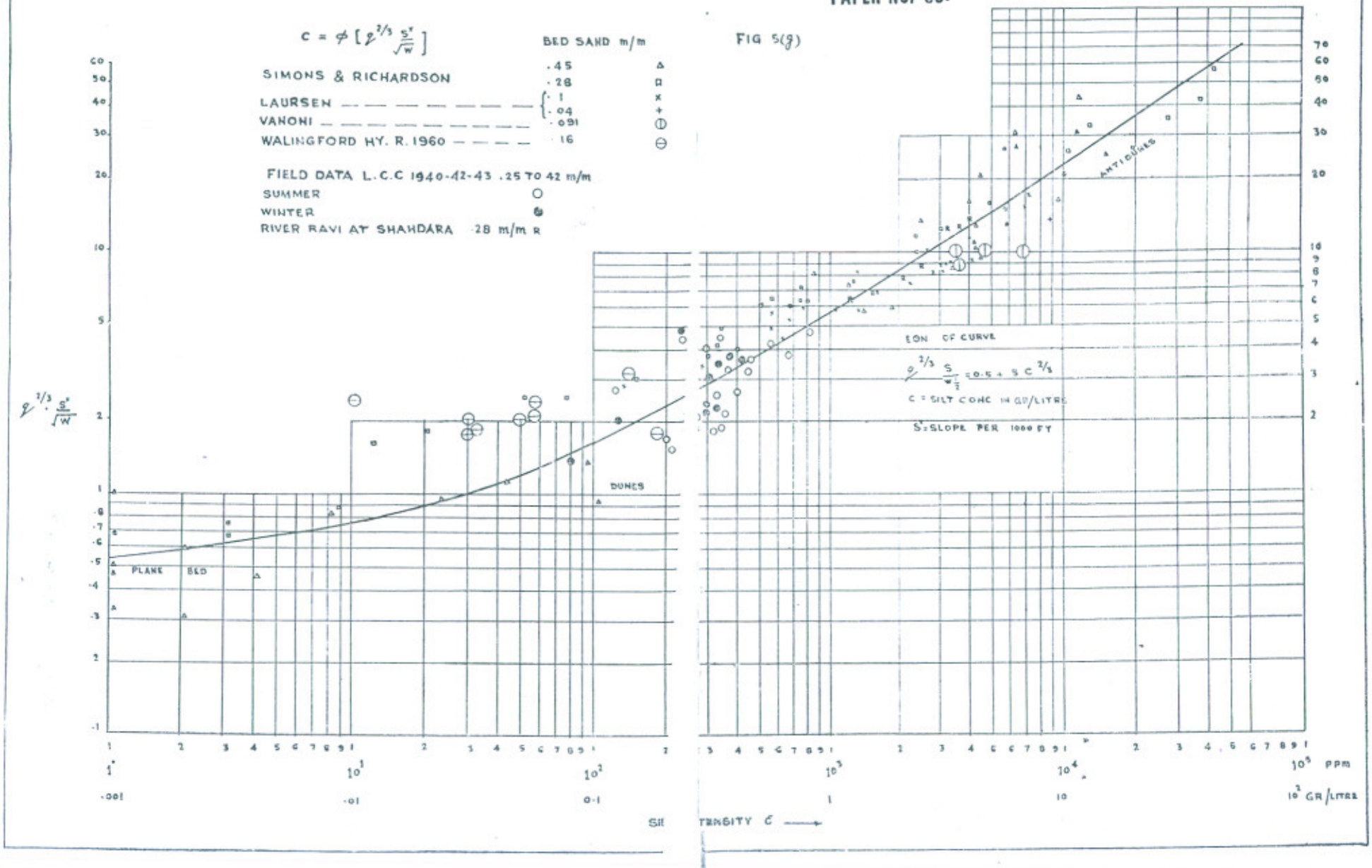
$$c = \phi\left(\rho^{2/3} \cdot \frac{gDs}{w}\right)$$




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FIG 5(g)


$C = \phi\left[g^{2/3}\frac{S'}{\sqrt{w}}\right]$
BED SAND m/m
SIMONS & RICHARDSON .45 Δ
.28 □
LAURSEN .1 ×
.04 +
VANONI .091
WALINGFORD HY. R. 1960 .16
FIELD DATA L.C.C 1940-42-43 .25 TO 42 m/m
SUMMER
WINTER
RIVER RAVI AT SHAHDARA .28 m/m R
$g^{2/3}\frac{S'}{\sqrt{w}}$
PLANE BED
DUNES
ANTIDUNES
EQN OF CURVE
$g^{2/3}\frac{S}{w^{1/2}} = 0.5 + 5C^{2/3}$
C = SILT CONC IN GD/LITRE
S = SLOPE PER 1000 FT
SILTENSITY C
ppm
GR/LITRE