PAPER No. 254

WATERWAY REQUIRED UNDER BRIDGES WITH SPECIAL REFERENCE TO THE ROHI BEAS BRIDGE ON NORTH WESTERN RAILWAY NEAR BEAS RAILWAY STATION

By Karnail Singh, Executive Engineer, and Gurdial Singh, Assistant Engineer, Projects and Design Branch, North Western Railway, Lahore

I.—Waterway for Bridges

1. *Introduction*.—The correct estimation of waterway required under a bridge is a problem which every engineer is confronted with many a time in his professional career. Recently a reference suggesting the reduction of waterway under the Rohi Beas Bridge to one third of its existing length was received for approval in the N. W. R. Headquarters Office. During the course of its check in the Projects and Designs Branch, which led to further investigations, the authors found to their regret that notwithstanding the fact that hundreds of streams have been bridged, diverted or blocked by the engineers in this province during the last decade or two, no recent contribution on the subject of "waterway required under bridges" in the form of an authoritative publication had been made by any one of those, who by virtue of their personal experience are more suited to tackle the problem. In the early days when the science of river engineering had not progressed far, the waterway was fixed in an arbitrary manner and there was a tendency for bridging the entire widths of streams. This practice not only meant larger initial capital cost but it also involved heavier recurring maintenance charges. Nearly half a century ago, valuable contributions on the subject of River training were made by Bell and later developed by Spring in his famous work. It was found that waterways under most of the bridges, built in the last century, could be reduced with advantage and it was actually done. As an instance, the Alexandra bridge over the Chenab at Wazirabad was originally a bridge of 64 spans, and by stages it has now come down to 17 spans only. Considering the magnitude of bridge work that has been done in this province and nearby, it is high time that certain definite lines of investigations on the subject were laid down so that a considerable waste of money involved in the bridging of extra widths and subsequent reductions of the waterway is avoided in future. This paper is not the result of any personal practical experience, but it is an attempt to explain the main considerations underlying the problem and how it was tackled in the particular case of the proposed reduction in waterway under the Rohi Beas Bridge. The authors fervently hope that the subject will elicit

sufficient interest from the profession and will afford an opportunity to the experienced engineers to express their views for the benefit of those, who may be called upon to deal with this problem in future.

2. *Classification of streams.*—For the purpose of bridgework, these can be classified as :—

(*i*) Artificial channels.

(*ii*) *Natural channels,* which are again subdivided into

(*a*) Flood nullahs and torrents, and

(*b*) Perennial streams.

To tackle the problem of waterway required under a bridge the first step is the determination of maximum discharge. In the case of artificial channels as the maximum discharge is always a known quantity and as the engineer also knows the velocity at which water can be allowed to flow through the bridge no difficulty is experienced in fixing the waterway for the bridges. For natural channels, however, the determination of maximum discharge and the waterway required to pass it is a problem and it is dealt with in this paper.

(*a*) *Maximum Discharge in Flood nullahs and torrents.*—The maximum discharge, through flood openings, where the streams are not well defined may be determined by various methods, a few of which are briefly stated below :—

(*i*) *Dickens', Ryve's and other similar formulæ.*—These formulæ are of the form $Q = CM^n$, where 'M' is the catchment area in square miles and 'C' and 'n' are constants. In Dickens' formula the value of 'n' is $\frac{3}{4}$ and in Ryves $\frac{2}{3}$. The value of 'C' in a particular case is hard to assign as it must depend on rainfall, nature of country, its shape, size and slope. Col. Dickens' assigned a value of 825 to 'C' for an annual rainfall of about 36 in. and held that it could be applied to localities having rainfall varying from 24 in. to 50 in. In Ryve's formula which is mainly used in Madras, the value of 'C' varies from 450 to 675. There are several other formulæ in use but none of these can be of general application, due to the variety of conditions on which the runoff depends.

(*ii*) *Dun's table.*—James Dun, an American Engineer made a notable contribution to this complex subject by giving the waterway required for catchment areas of different sizes both for hilly and plain countries in the form of a table, which can be of considerable use to an engineer, if judiciously employed. The table will be found on page 1112 of Waddell's Bridge Engineering and also as table VIII in M. E. S. Handbook Vol. III. This is reproduced as Appendix "A" of this paper.

(*iii*) *Waddel's graph.*—An extensive study of rainfall and its runoff was made by the American Railway Engineering Association and a digest of those investigations is given in the form of a graph by Waddel on page 1117 of his famous work on "Bridge Engineering." This is reproduced as Appendix "B" of this paper.

(*iv*) *Hydrograph method.*—A more exact method of determining the discharge through a flood opening can only be through actual observations of the intensity of rainfall in the catchment area. Unfortunately the records of rainfall available in this country do not give any figures for the intensity of rainfall and are not thus very useful for determining the maximum runoff. What is required is a continu us record and its details during the period of maximum concentration. These records can only be made available by the installation of automatic recording rain gauges, which do not appear to be a possibility in this country for some time yet.

Knowing the maximum intensity of rainfall over an area, it becomes necessary to estimate the period of maximum concentration, which is influenced by the time taken for the water to flow from the limits of the area under consideration to the point of "discharge." This time is the total time taken in reaching the stream and then in the stream to the point of discharge. It is evident that the larger the catchment area, the more will be the time taken by the rain water to reach the point of discharge and less will be the intensity of rainfall to be allowed for larger catchment areas and *vice versa*. The exact treatment of determining the discharge from a large area having several rain gauges is given in the Proceedings of the American Society of Civil Engineers for November, 1938.

(*v*) *Lloyd-Davies' Method.*—The flow-off from a Catchment naturally depends on (1) its area (2) the character of surface (3) the slopes of the ground, and (4) the intensity of rainfall during the period of concentration. The whole of the rain falling over an area does not find its way to the drainage lines and some of it is lost through (1) sucking up by vegetation (2) percolation into the ground, unless the soil is already saturated (3) retention in natural fissures, basins, and hollows and (4) evaporation. The maximum flow-off will naturally occur when the soil is already saturated. In this treatment, the discharge from a catchment area is expressed by the formula $Q=(60{\cdot}5\times 60\times r)\times A\times p.$,

Where Q = Discharge in C.ft. per minute.

r = Total rainfall in inches during the time of concentration.

T = Time of concentration including time of entry in minutes.

A = Area in acres.

p = Proportion of total rain fall which flows into the drainage lines after making allowance for all losses. The value of "p" differs very widely. For lawns, gardens and meadows it may be taken from ·05 to 0·25 and for wooded areas from ·01 to 0·2. This method is described in detail in Bevin and Ree's book on Sewerage.

(*vi*) *Khangar and Gulati's method.*—Following the lines indicated in (*v*) above, an able adaptation for the determination of discharges in the Punjab is given in the paper No. 178 on Sillanwalli Drain read before this Congress in 1935.

(vii) *By comparison with observed discharges under similar conditions.*—Sometimes an engineer may be fortunate to take advantage of the observations on an existing bridge for the same stream or any other under similar conditions. The tables and data given on pages of Buckley's Irrigation Pocket Book, will also be found useful for such comparison.

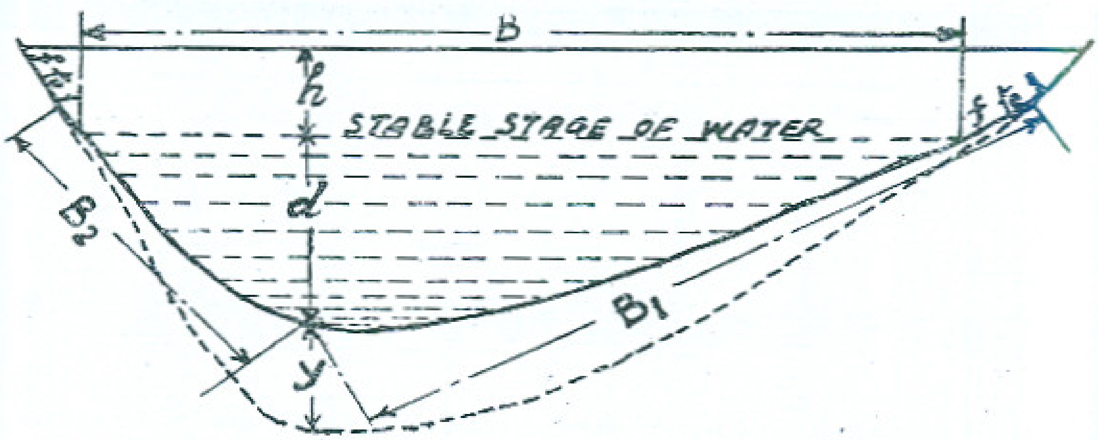
(b) *Maximum Discharge in Perennial streams.*—Most of the rivers and streams in this part of the country have been bridged over at many places and several of them have weirs built across them. The determination of waterway for another bridge on any one of these can be more easily determined by inference without going into 'ab initio' investigations. As a check and to meet instances where such references are not available, the discharge can be determined by the methods described under (a) Flood nullahs above, though such estimates are not considered reliable enough for important bridges. In those cases, it is advisable to determine the discharge either by actual measurement, or by the application of formulæ on the flow of water in channels.

The measurement of discharge during a flood is by no means an easy matter, since it is difficult to take reliable soundings especially for streams, where scour is deep. The usual method however is (i) to divide the width into a number of compartments, (ii) to determine its depth by taking soundings in each and (iii) to find out the velocity by means of floats between two selected sections. A current meter is also useful for the purpose. Knowing the area and velocity, the calculation of discharge is a simple matter. The mean velocity is usually taken to be 0·86 of the surface velocity.

For the determination of discharge by the formulæ applicable to open channels, the procedure is as follows :—

(i) Locate the H. F. L. line from flood marks or by local enquiry (ii) make a careful survey of the cross-section of the river below this flood line and plot it on a graph paper (iii) determine the slope of water line or the bed of the stream (iv) calculate the cross-sectional area and wetted perimeter of the stream upto the High Flood Line. If the area is denoted by A and the wetted perimeter by P, the hydraulic mean radius $r = \frac{A}{P}$ — (v) determine the mean velocity by Chezy's formula $V = C\sqrt{rS}$ where S is the slope per ft. length. The bottom velocity may be taken as $\frac{5}{6}V$. In the silt carrying rivers of India, it is usually found that this velocity is higher than the critical velocity of scour. Thus the bed of the river will be cut till the velocity is brought down to the non-scouring critical mean velocity, which is usually expressed by the formula $V_s = cd^m$, where 'c' is a constant. The value of 'm' depends on the type of the material in the bed and "d" is the depth of water. According to Kennedy's theory V_o may be taken as $0{\cdot}84d^{0{\cdot}64}$ for the silt, usually found in the Punjab streams. This corresponds to a bottom velocity $V_s = \frac{5}{6} \times 0{\cdot}84d^{0{\cdot}64} = 0{\cdot}7d^{0{\cdot}64}$.

There is a certain stage of water level in the river at which no scour takes place and the bed is stable. This stage can be determined by observation or calculation. When the water rises above this level, scour takes place. A mathematical relationship which exists between the rise of water level and the corresponding depth of scour is established hereunder. It is evident that scour can occur only when the mean velocity in the whole cross-section rises as a result of the rise in water level. Assuming the slope of water surface to remain constant, the velocity can only increase by increase in the hydraulic mean radius *i.e.*, the ratio $\frac{A}{p}$. This can only happen if 'A' increases at a faster rate than 'p' or when $\frac{\Delta A}{A}$ is greater than $\frac{\Delta P}{P}$. It is thus clear that under equilibrium conditions, the cross-section of the channel will tend to assume a form in which its perimeter is a minimum for a given area, to achieve which the bed of channel after scour must assume a curved form.



Assuming a rise 'h' in the water surface to have caused a scour 'y' the total area of the cross-section and the total perimeter will be found as follows :—

Increment of area at top $= h\left(B + \frac{fh}{2} + \frac{f'h}{2}\right)$

Increment of area at bottom assuming the curve taken after scour as parabolic $= \frac{2}{3} B_1.y + \frac{2}{3} B_2\, y = \frac{2}{3}(B_1 + B_2)\, y$.

$$\text{Total increment in area} = h\left(B + \frac{h}{2}(f + f')\right) + \frac{2}{3}(B_1 + B_2)\, y \qquad \ldots \quad \ldots \quad \ldots \quad (1)$$

Increase in wetted perimeter at top $= h\sqrt{1+f^2} + h\sqrt{1+f'^2}$

Increase in wetted perimeter at bottom :—

Length of parabolic arc on right =

$$B_1 \quad \left(1+\frac{8y^2}{4\times 3B_1{}^2}\right) \text{ app.} = B_1+\tfrac{2}{3}\frac{y^2}{B_1}$$

Similarly length of parabolic arc on left $=B_2+\frac{2}{3} \cdot \frac{y^2}{B_2}$

$\therefore$ Total increase in perimeter at bottom

$$=\frac{2y^2}{3}\left(\frac{1}{B_1}+\frac{1}{B_2}\right) = \frac{2y^2}{3} \cdot \frac{B_1+B_2}{B_1 \cdot B_2}$$

$\therefore$ Total for top and bottom $= h\left(\sqrt{1+f^2}+\sqrt{1+f'^2}\right)+$

$$\tfrac{2}{3}\, y^2\, \frac{B_1+B_2}{B_1 . B_2} \qquad .. \qquad .. \qquad (2)$$

Denoting the original area by A and the wetted perimeter by 'P', the new hydraulic mean radius will be

$$\frac{A+h\left(B+\frac{h}{2}(f+f')\right)+\frac{2}{3}(B_1+B_2)\,y}{P+h\left(\sqrt{1+f^2}+\sqrt{1+f'^2}\right)+\frac{2}{3}\,y^2 . \frac{P}{B_1 B_2}} \qquad .. \qquad (3)$$

$= r+\triangle r$. But $r = \frac{A}{P}$. Deducting this from equation (3) the value of $\triangle r$ is easily determined.

The velocity in a channel is expressed by the well known formula $v = c\sqrt{rS}$. Applying Kutters' formula

$$C = \left(\frac{\frac{1\cdot811}{n}+41\cdot6+\frac{\cdot00281}{S}}{1+\left(41\cdot6+\frac{\cdot00281}{S}\right)\frac{n}{\sqrt{r}}}\right), \quad \text{where n is the}$$

coefficient of rugosity, S the slope and r the hydraulic mean radius. It will be seen that the numerator in this expression is a constant since 'S' is assumed to be constant and may be denoted by K. In the denominator the term $\left(41\cdot6+\frac{\cdot00281}{S}\right)n$ is a constant and will be denoted by a.

$$\text{Then } C = \frac{K}{1+\frac{a}{\sqrt{r}}} = \frac{K\sqrt{r}}{\sqrt{r}+a} . \qquad \therefore \quad V = \frac{K\sqrt{r}}{\sqrt{r}+a} . \sqrt{rS}$$

$$= \frac{Kr}{\sqrt{r}+a}\sqrt{S} \qquad .. \qquad .. \qquad .. \qquad (4)$$

Differentiating 4 with reference to 'r'

$$\frac{\triangle V}{\triangle r} = \frac{(\sqrt{r}+a)\ K\sqrt{S}\ \ Kr\sqrt{S}\ \ (\tfrac{1}{2}r)^{-\frac{1}{2}}}{(\sqrt{r}+a)^2} =$$

$$\frac{K\sqrt{S}\ (\tfrac{1}{2}\sqrt{r}+a)}{r+a^2+2a\sqrt{r}}$$, which gives

$$\triangle r = \frac{r+a^2+2a\sqrt{r}}{K\sqrt{S}\ (\tfrac{1}{2}\sqrt{r}+a)} \cdot \triangle v. \quad \ldots \quad \ldots \quad (5)$$

$\triangle v$ is the increase in mean velocity and as already stated the increase in bottom velocity causing scour may be taken 5/6 of this or

$\triangle vs = 5/6\ \ v.$

Now $v^s = md^{0\cdot64}$. Differentiating this with reference to

$$d\ \frac{\triangle vs}{\triangle d} = 0\cdot64\ md^{-\cdot36}$$

or $\triangle vs = 0\cdot64\, md^{-\cdot36} \triangle d$

When the equilibrium is established

$\triangle d = y+h$ and hence $\triangle vs = 0\cdot64\ md^{-\cdot36}(y+h)$

$$\therefore\ \triangle v = \frac{6}{5} \times 0\cdot64 md^{-\cdot36}\ (y+h).$$

Substituting this value of $\triangle V$ in equation 5 :—

$$\triangle r = \frac{r+a^2+2a\sqrt{r}}{K\sqrt{S}(\tfrac{1}{2}\sqrt{r}+a)} \times \frac{6}{5} \times \frac{0\cdot64m}{d^{0\cdot36}}\quad (h+y)$$

$$\therefore\ r+\triangle r = \frac{A}{P} + \frac{r+a^2+2a\sqrt{r}}{K\sqrt{S}(\tfrac{1}{2}\sqrt{r}+a)} \times \frac{\cdot77m}{d^{\cdot036}}\ (h+y). \quad \ldots \quad \ldots \quad (6)$$

Equating this value of r with that found in equation (3) we have :—

$$\frac{A+h\ (B+\frac{h}{2}\ (f+f')+\tfrac{2}{3}\ (B_1+B_2)\ y}{P+h\ (\sqrt{1+f^2}+\sqrt{1+f'^2}) + \tfrac{2}{3}\ y^2.\ \dfrac{P}{B_1 \times B_2}}$$

$$= \frac{A}{P} + \frac{r+a^2+2a\sqrt{r}}{K\sqrt{S}\ (\tfrac{1}{2}\sqrt{r}+a)} \times \frac{\cdot77m}{d^{0\cdot36}}\quad (h+y) \quad \ldots \quad (7)$$

This is a cubic equation in 'y' and can be solved either by the method of solving cubic equations (*vide* Advanced Practical Mathematics by Cowley pages 91 and 92), or by trial and error. The first solution may be found by dropping the term containing y^2, as the fraction $\tfrac{2}{3}.\ \dfrac{P}{B_1.B_2}$, which is its coefficient will be usually small.

Having determined the depth 'y', it is simple to determine the maximum discharge. The above method has been given in detail with the hope that it will be of use in estimating the discharges of rivers during floods, which cannot be correctly found by the ordinary method applicable to non-scouring beds.

4. *The length of waterway required for a Bridge.*—Having determined the maximum discharge, the next step is to ascertain the length of Bridge required to pass the whole or part of it satisfactorily. Unless otherwise necessitated for considerations of safety, it is evidently uneconomical to provide a bridge for passing the highest flood, for such floods are rare and occur after very long intervals. Even then the costs incurred in repairing the damage done will be offset by savings on account of interest on the additional initial cost and maintenance charges for the provision of a longer bridge. No hard and fast rules can, however, be laid down but it is incumbent on the engineer to exercise his judgment keeping in view both the engineering and economic factors.

In the plains of India the main current of all alluvial rivers meanders within a fairly wide strip of land and the bridging of the whole width can only be done at a prohibitive cost. It is usually sufficient to build a bridge of a determined width and train the river waters to flow through it. The whole subject of the required training works is fully dealt with in Spring's "River Training and Control" (Technical Paper No. 153), which it will pay every engineer to read once and over again. The question of the width of bridging is intimately connected with the regime of stable channels in alluvium, which subject has been ably discussed by Lacey in Vol. 229 of the Minutes of Proceedings of the Institution of Civil Engineers. Briefly stated the conclusions arrived at by him are as follows :—

(*i*) To pass a certain discharge there is a fixed cross-section for a channel, which can be determined mathematically.

(*ii*) In all regime channels $Af = A'f' = A$ "f" where A, A' etc., are the areas of cross-section and f, f', etc., are silt factors. The value of 'f' is determined from the equation $f = \left(\frac{V}{V_o}\right)^2$, where V is the actual critical velocity and 'V_o' the velocity by Kennedy's formula $V_o = 0{\cdot}84\ d^{0{\cdot}64}$.

(*iii*) The critical velocity for all regime channels is determined by the formula $V_o = 1{\cdot}17\sqrt{fR}$, where 'f' is the silt factor already referred to and R is the hydraulic mean radius.

(*iv*) In all silt transporting channels $Qf^2 = 3{\cdot}8V_o{}^6$, which fixes a definite relationship between the discharge and velocity.

(*v*) All natural silt transporting channels have a tendency to assume a semi-elliptical cross-section.

Knowing the discharge and silt factor for a stream, the velocity is determined from the formula $Qf^2 = 3{\cdot}8V_o^6$ and $\frac{Q}{V_o}$ gives the cross-section area. The hydraulic mean radius is determined from the formula $V_o = 1{\cdot}17\sqrt{fR}$. Knowing A and R, an unique cross-section is easily determined for the channel. These formulæ can be of help in checking the work described above for the determination of discharge.

Mr. Lacey has derived a very remarkable formula connecting the wetted perimeter with the discharge independent of the silt factor. This formula is $P_w = 2{\cdot}668\ Q^{0\cdot5}$. In large rivers the wetted perimeter is practically the same as the width of waterway and hence the minimum width of stable waterway may be taken $W_s = 2{\cdot}67\ Q^{0\cdot5}$. This formula has been tested against the actual waterway provided for important bridges and gives the results in close conformity with practice. It may be taken as a very good rule for fixing the required waterway. The remaining width of the river, can be blocked and guide banks provided for training the channel to flow under the bridge.

II.—The Rohi Beas Bridge

1. *History.*—This bridge is at mile 277 between Beas and Butari on the main line of the N. W. R. over a flood channel, Rohi Beas, and has 3 spans of 100 ft. each. The original bridge, which now carried only a siding, was constructed before 1867 when the railway line between Beas and Amritsar was first built. No records are available to show as to why a waterway of 3—100 ft. spans was provided. In 1910 the Railway line was doubled and from the reports in the Project of this work it appears that the new bridge was also provided with 3—100 ft. spans simply because the old bridge had this waterway. The waterway under this bridge was noticed to be excessive, and after some investigations the Divisional Superintendent, Lahore, recommended the retention of one span only in place of three. The authors had to examine the proposals in the N. W. R. Headquarters Office before a decision could be given by the Chief Engineer and the details of the study so far made of this problem are given below.

2. *Geography.*—The word 'Rohi' simply means a drainage line and the 'Rohi' under reference is the most easterly drainage line of the Amritsar district. It is also known as the Riarki Vaug. West of this there is a stretch of fertile country 6 to 8 miles broad, irrigated by the Sabraon branch of the Upper Bari Doab Canal. The following extract from the Gazetteer of the Amritsar district gives a clear idea of the waters in such Rohis. "Water only flows along this flood line at intervals of several years, after exceptionally heavy rain, and the line consists of a broad shallow depression, marked on both its edges by a strip of sandy soil, sometimes forming into shifting sand hills, but more usually taking the form of undulating slopes which are sown with crops of wheat and grain, jowar and pulses. The chance of flood is so small that the whole is sown even to the centre of the

depression. Floods (as in 1875) have been known to do considerable damage to the land lying in the track of this line, choking up wells with the sand brought down, and going near to wrecking villages within its influence. But in an ordinary year, the depression is so shallow and indistinct, and cultivation so general, in and on the edges of the line, that all that would be noted by a casual observer crossing it, would be that the ground had changed from level to undulating, that trees were scanty, and the soil was sandy instead of the usual light loam."

The Rohi under reference derives the whole of its discharge from the rain on its catchment basin and there is no possibility of any water from the Beas entering into it, because the right bank of the river is a continuous cliff 20 to 30 ft. high, the upper part of which is hard clay mixed with kankar and the lower stratum usually fine river sand. The left bank, on the other hand, is low and liable to inundation.

About sixty or seventy years ago an attempt was made to divert the water of the Rohi into the Beas River about half a mile above the bridge. A bund was constructed across the Rohi but it did not stand and the diversion cut into the Beas never functioned.

3. *The Main Issues of the Problem.*—For a systematic study and investigation the following issues suggest themselves :—

(*a*) What were the reasons for providing three spans of 100 ft. each for this bridge originally ?

(*b*) Have the conditions in the catchment area of this waterway altered since the bridge was originally built ? In other words, were the Irrigation Works now in existence upstream also there at the time of construction and, if not, what effects their construction has produced on the flood discharge through this bridge ?

(*c*) What is the maximum discharge that can be expected now ?

(*d*) What useful information can be gathered regarding the waterway provided on bridges nearby ?

(*e*) If one or two spans can be closed, will the cost of closing a part of the bridge involving as it does the strengthening of one or two piers to act as abutments be financially justified ?

These points are dealt with *ad seriatim* below :—

(*a*) *The waterway under the old Bridge.*—As mentioned before, the waterway under the bridge constructed before 1867 consisted of three spans of 100 ft. The reasons for adopting this waterway are not known, but it appears that it was in conformity with the practice of ancient engineers to bridge the entire widths of the flood nullahs. The present bridge has the same waterway as the previous one. The piers and abutments are founded on a bed of clay and sand about 50 ft. below the bed of the Rohi. It shows that the engineers responsible for the construction of this bridge expected a deep scour, although from the present estimates of discharge detailed below a very deep scour does not appear feasible.

(*b*) *Effect of Irrigation works on Discharge.*—The Upper Bari Doab Canal was opened in 1859 and the Irrigation commenced in the following year. The exact date of opening of the Sabraon Branch could not be ascertained but it was probably soon after that of the main canal. If so, most of the irrigation channels must have been in existence at the time of the construction of the Railway. Even if this branch and its connected works were constructed subsequently, the catchment area of the Rohi could not have been appreciably affected. The fundamental principle behind the construction of all canals and distributaries is that they either lie on or are very near the water shed lines. As such these do not seriously effect the discharge of streams which naturally follow the depression lines. If anything, the effect of irrigation should be to increase the runoff due to a greater saturation of the tract. We may conclude therefore, that the probable discharge in the Rohi is the same now as it was, when the Railway bridge was first constructed.

(*c*) *The Probable Discharge of the Rohi at the Site of the Bridge.*—The catchment area for the Rohi Beas is 84 square miles. The probable discharge from this area as determined by the various methods and the waterway required is as follows :—

(*i*) *Dicken's formula* :—

$$Q = CM^{\frac{3}{4}} = 825 \times 84^{\frac{3}{4}} = 22900 \text{ cusecs.}$$

Width of waterway required according to Lacey's formula
$= 2.67 \times (22900)^{0.5} = 404$ ft. length.

(*ii*) *Waterway area from Dun's Table.*—From table VIII in Military Works Handbook Vol. III, the area required for waterway in plains assuming 50% runoff = 973 sq. ft. Assuming a velocity of 3 ft. per sec. discharge = 2919 c.ft. per second.

Width of waterway required $= 2{\cdot}67 \times (2919)^{0{\cdot}5}$
$= 2{\cdot}67 = 54{\cdot}03 = 144$ ft.

$V_o = \left(\frac{2919}{3{\cdot}8}\right)^{1/6} = 3{\cdot}03$ ft. per sec, which checks the figure assumed.

(*iii*) *Runoff from Weddell's graph.*—The runoffs are given for an average annual rainfall of 35 in. In this case of 23·5 inch rainfall, the discharge may be taken as $\frac{40 \times 23{\cdot}5}{35} = \frac{940}{35} = 27$ cusecs, per sq. mile giving a total discharge of 2268 cusecs for 84 sq. miles.

Width of waterway required by Lacey's formula $= 2{\cdot}67 \times (2268)^{0{\cdot}5}$ $= 127$ ft. During years of heavy flood this may be 254 ft.

(*iv*) *Hydrograph method.*—Not applied for want of data.

(*v*) *Lloyd-Davies' Method.*—The formula given on page 4 above for discharge is :—$Q = \left(60{\cdot}5 \times \frac{60}{T} \times r\right) \times Ap = 60{\cdot}5\ RAp$, where 'R' is the rainfall in inches per hour and 'A' the catchment area in acres. As mentioned before the rainfall intensity per hour is not known but may be taken 0·5 in. as a good guess. Then $Q = 60{\cdot}5 \times \frac{1}{2} \times 640 = 1626240$ p. c. ft. per minute, p may be taken ·05, therefore $Q = 81312$ c.ft. per minute or 1355 cusecs. Waterway required $= 2{\cdot}67\,(1355)^{0{\cdot}5} = 98$ ft.

(*vi*) *Khangar and Gulati's method.*—Not applied for want of data.

(*vii*) *Table of Discharges in Buckley's Irrigation Pocket Book.*—On p. 349 of the above book a table of runoffs is given which was used in determining waterway under canals and distributaries and it is stated that similar values were used for the Upper Chenab Canal. In this table for an average annual rainfall of 32 in. the discharge per sq. miles is given as 26 cusecs. The average annual rainfall for the Amritsar Tehsil is $23\frac{1}{2}$ in. (The heaviest fall recorded was 47 in. in 1908-1909). The average run off per sq. mile may therefore be taken as $\frac{26 \times 23{\cdot}5}{32} = 19$ cusecs. Then total discharge $= 84 \times 19 = 1596$ c.ft. per sec. and the width of waterway required by Lacey's formula $= 2{\cdot}67 \times (1596)^{0{\cdot}5} = 107$ ft. For the heaviest fall of 47 in. however, this width may be as 214 ft.

(*d*) At a short distance below the Railway bridge the Rohi-Beas crosses the Grand Trunk Road over a Causeway. The width of water surface at the top is about 300 ft. and the maximum depth of water 2′-6″. Assuming the curve to be parabolic, area of waterway $= 300 \times \frac{2}{3} \times 2{\cdot}5 = 500$ sq. ft. The velocity at the site is not known but it is stated that no traffic can pass during floods. This velocity is judged to be about 3·5 ft./sec. for this depth of water. As an interesting check, however, a rough estimate of the velocity may be formed by calculating its magnitude which would carry a man off his feet. Momentum destroyed per second gives the water pressure and is equal to $\frac{62{\cdot}5\ V^2}{32} = 1{\cdot}95\ V^2$. Assume the exposed area of man to be 2′-6″ × 12″ or 2·5 sq. ft. Then total pressure $= 2{\cdot}5 \times 1{\cdot}95\ V^2 = 4{\cdot}875\ V^2$. The average weight of a man may be taken 150 lbs. and the coefficient of friction 0·4 giving a frictional force of 60 lbs. Hence the velocity is roughly given by the equation $4{\cdot}875\ V^2 = 60$, whence $V = 3{\cdot}5$ ft. per sec. The discharge is then $500 \times 3{\cdot}5 = 1750$ cusecs. Waterway required $= 2{\cdot}67\,(1750)^{0{\cdot}5} = 112$ ft.

Conclusions for maximum discharge and waterway required etc.—Against the calculated discharge of 22900 cusecs by Dicken's formula, the figures by other methods vary between 1355 and 2919 cusecs, and the corresponding widths of waterway required range from 98 to 144 ft. Rounded figures of 3000 cusecs for the discharge and 150 ft. as the width of waterway required is therefore considered adequate.

Mean velocity from Lacey's formula $Qf^2 = 3{\cdot}8\ V_0$ assuming 'f' to be unity works out to 3·04 ft./sec.

Similarly value of hydraulic mean radius 'R' from Lacey's formula $V_o=1{\cdot}17\sqrt{fR}$ works out 6·75. The maximum depth of water in this case of straight reach will thus be $1{\cdot}27R=8{\cdot}57$ ft. according to Lacey.

It follows therefore that two spans out of three existing 100 ft. ones should be retained and only one span should be closed.

(*e*) *Financial Implications of the Problem.*—The closing of one span of the bridge will necessitate the strengthening of one pier to act as abutment, which can be done by either—

(*i*) Building a retaining wall behind the pier to carry all earth pressure and also providing new wing walls, or (*ii*) by filling boulders or broken stone to a slope 1 to 1 towards the top at the back of the pier to relieve the earth pressure on the pier. Wing walls to be provided for retaining the side slopes.

Or (*iii*) by making the pier into a buried abutment *i.e.*, to fill the back of the abutment with boulders and let the filling flow in front thus doing away with the necessity of providing wing walls etc.

Out of the above three alternatives, item (*iii*) costing Rs. 2,000 is found to be the cheapest.

The cost of earth filling in the span to be closed and the necessary pitching is estimated to be Rs. 3,000.

Against the expenditure of Rs. 5,000 on the closing of one span a credit of Rs. 11,000 being the value of girders to be released after allowing for dismantling the temporary arrangements, and carriage charges etc., is expected to be realised.

In addition to the above noted saving of Rs. 6,000, there will be a saving of approximately Rs. 6,000 per annum on account of reductions in (*i*) maintenance charges (*ii*) contribution to Depreciation Fund and general revenues, and (*iii*) interest charges on the capital cost.

Appendix 'A'

Waterway Areas Required for Catchment Areas.

Based on the Dun Drainage Table.

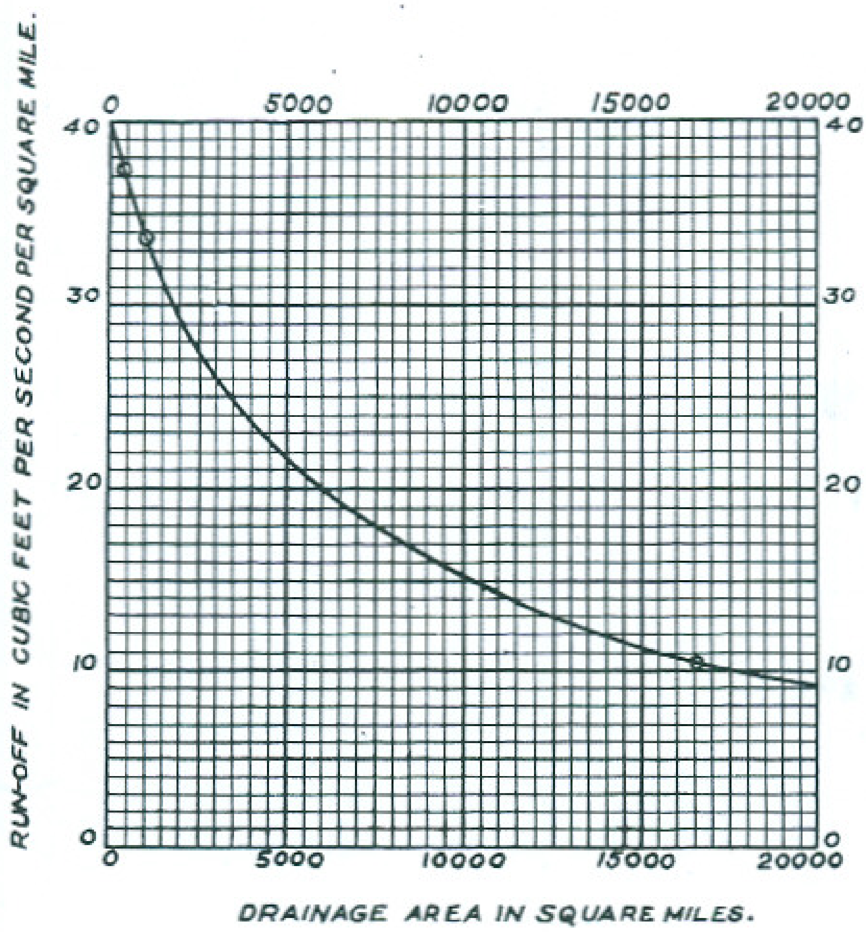
Catchment Area in Sq. Miles.	Area of Waterway in Sq. Ft.				Catchment Area in Sq. Miles.	Area of Waterway in Sq. Ft.			
	In Hills.		In Plains.			In Hills.		In Plains.	
	120%	100%	80%	50%		120%	100%	80%	50%
·01	2·4	2·0	1·6	1·0	12	888	740	592	370
·02	4·8	4·0	3·2	2·0	14	966	805	664	403
·04	9·0	7·5	6·0	3·8	16	1038	865	692	433
·06	12·6	10·5	8·4	5·3	18	1104	920	736	460
·08	16·2	13·5	10·8	6·8	20	1164	970	776	485
·10	19	16	13	8	25	1296	1080	864	540
·20	38	32	26	16	30	1416	1080	944	590
·30	53	44	35	22	35	1528	1273	1018	637
·40	67	56	45	28	40	1620	1350	1080	675
·50	79	66	53	33	50	1812	1510	1208	755
·60	89	74	59	37	60	1980	1650	1320	825
·80	106	88	70	44	70	2136	1780	1424	890
1·0	120	100	80	50	80	2280	1900	1520	950
1·2	144	120	96	60	90	2418	2015	1612	1008
1·4	168	140	112	70	100	2544	2120	1696	1060
1·6	192	160	128	80	120	2778	2315	1852	1158
1·8	216	180	144	90	140	3000	2500	2000	1250
2·0	240	200	160	100	160	3198	2665	2132	1333
2·5	300	250	200	125	180	3384	2820	2256	1410
3·0	360	300	240	150	200	3564	2970	2376	1485
3·5	419	349	279	175	250	3970	3308	2646	1654
4·0	466	388	310	194	300	4338	3615	2892	1808
4·5	509	424	339	212	400	4998	4165	3332	2083
5	546	455	364	228	500	5532	4610	3688	2305
6	611	509	407	255	600	6036	5030	4024	2515
7	667	556	445	278	700	6504	5420	4336	2710
8	721	601	481	301	800	6960	5800	4640	2900
9	769	641	513	321	900	7296	6080	4864	3030
10	815	679	543	340	1000	7656	6380	5104	3190

APPENDIX 'B'

PAPER NO. 254.

AVERAGE RUN-OFFS FOR THE UNITED STATES.

(COPIED FROM BRIDGE ENGINEERING BY J.A.L WADDELL PAGE 117)



PUNJAB ENGINEERING CONGRESS
1942.

DISCUSSION

S. Karnail Singh, one of the authors while introducing his paper, remarked that the paper was the outcome of a practical difficulty, which the authors had been confronted with in the normal performance of their duties, and in the few pages of the paper they had attempted to put forth all that they could get at from different sources for the working out of the problem, with the one and only hope that other engineers who had to tackle a similar proposition in future can usefully refer to this paper and avoid wading through a lot of scattered literature on the subject. That was thus his only apology for presenting the paper.

2. No doubt there was a mass of literature on the subject already available in different shapes and at different places, but there was no authoritative treatment of the same in a clear and concise manner which could be readily referred to. Furthermore, the results obtained according to different treatments were so diverse that one was apt to give up all hopes for continuing the theoretical analysis of the problem. Although the author might be charged as being a cynic, or something similar, he ventured to say that, more often than not, an engineer faced with the problem of fixing a suitable waterway for a flood opening made up his mind on the field on what he judged to be the actual requirement and then started to find out reasons in support of his pre-conceived decision. To illustrate his statement the speaker then narrated an example or two from his own experience.

First, in the light of his observations and the knowledge of the topography of the country, one engineer decided that a waterway of 250 feet would suffice for the particular flood opening. He, then, went on to support his decision by adding that the catchment area for the opening being 100 square miles, his proposal of 250 feet waterway was in accordance with the results obtained by the application of Dicken's formula. He wrote down thus :

> "Maximum Expected Discharge$=CM^{3}/_{4}=825\times100^{3}/_{4}=2609$ Cusecs Allowing a depth of $3\frac{1}{2}$ feet of water flowing at a velocity of 3 feet per second, the required water-way is equal to 250 L.ft."

When it was pointed out to him that the mathematical result of his equation would be "26090", and not "2609" as written down by him, the engineer turned *volte face* and emphatically stated that the Dicken's formula was not applicable in that particular case and that the Punjab Irrigation formula, *i.e.* Discharge$=5\sqrt{A}$, where "A" is catchment area in square miles, would be the appropriate one.

The other instance that he had come across was the application of Dun's Tables for the evaluation of discharge in an another case, wherein the engineer concerned observed that "the topography of the country being more or less the same as that of the Dun Valley, the application of Dun's Tables would be the most correct solution". When told that Dun's Tables were named after James Dun, an engineer in the U.S.A., and not after the beautiful Dehra Dun Valley, he searched for something else. The foregoing instances went to show that the lack of a synopsis of the available literature on the subject was likely to lead some engineers at any rate into indefinite, vague and, might be, inexact investigations.

As stated in the printed Introduction on this paper, the authors did not claim any research work on the subject but they had endeavoured to collect the relevant available information in a concise form, which would be of use to an engineer actually employed on the work. Of course, the usefulness of this paper would be greatly augmented if the senior engineers in the Congress would please contribute towards its usefulness by placing their ideas on the subject before the other members.

In the manuscript form, when this paper had been gone through by Rai Bahadur A. N. Khosla, he advised that instead of referring to the other literature in the paper it would be advisable to append the same with it for ready reference. We are grateful to him for his advice and have accordingly appended one table and another graph, which could be so done, without adding to the bulk of this paper.

The author said that perhaps he owed to the members another explanation for the meagre get-up of this paper, which unlike the fat bulk of the rest of them was a thin pamphlet of about a dozen pages only. As originally shaped, this paper looked quite presentable with a map of the country around Beas, the drawings of the Railway bridge and certain other details. But for obvious reasons those plans, etc., could not be published in print in these days of emergency and the result was an unadorned small pamphlet.

Before closing, the author pointed out that there were a few printing mistakes and omissions in this paper, and those, which really matter, were the following:

(1) *Page* 121, *at the end of sub-para* (*v*) "Lloyd-Davies' Method: $Q=(60{\cdot}5\times60\times r)\times A\times p.$", *should read* as

$$\frac{(60{\cdot}5\times60\times r)}{T}\times A\times P.$$

(2) *Page* 121, *at the end of sub-para* (*vi*), please *add:*
"Messrs. Khangar and Gulati have made another valuable contribution on the subject in, their paper No. 245 on 'Rinfall Runoff' read before this Congress last year."

(3) *Page* 125, *the second line on top should be:*

$$\frac{\triangle V}{\triangle r}=\frac{(\sqrt{r}+a)\,K\,\sqrt{s}-Kr\sqrt{s}.\,\frac{1}{2}\,(r)^{\frac{1}{2}}}{(\sqrt{r}+a)^2}$$

(4) *Page* 130 *in last line but one*, V_0 *should be* V_0^6.

(5) Graph at Appendix 'B' is based on an annual rainfall of 35 inches.

Dr. J. K. Malhotra congratulated the authors on their very creditable attempt to bridge a gap in the existing knowledge regarding the waterway required under bridges. They had modestly disclaimed any personal experience of the subject, but they had brought together in this paper a lot of useful information, which was likely to assist the designer considerably.

On page 119 the authors had stated that the waterways under most of the bridges built in the last century could be reduced with advantage, and as an instance, had given the case of the Alexandra bridge over the Chenab at Wazirabad.

In this connection the speaker would like to draw their attention to Fig. 13 on page 276 of Mr. Lacey's paper on 'Stable Channels in Alluvium'. In that diagram Mr. Lacey had plotted the minimum stable waterway against the maximum flood discharge, to a logarithmic scale. He had shown that the spans of the Alexandra bridge had been gradually reduced from 64 to 17, and that the final width as fixed in 1917 very closely fits his equation for minimum width of stable waterway of large rivers in alluvium, namely $W_S = 2.67\ Q^{0 \cdot 5}$. The following lines might be quoted from Mr. Lacey's paper: "The points for the Alexandra Bridge across the Chenab, read with the dates, are illuminating. When the bridge was constructed in 1876 it was given no less than 64 spans of 132 feet clear, the old bridging principle being that of bridging very nearly everything that presented the appearance of a river bed. The other points represent successively the engineer's tentative efforts to confine the river to one stable waterway. In 1917 it was realized that this river when in flood had never made effective use of more than 17 spans, and to that number of spans the bridge was accordingly cut down as a final measure of control." (The other points referred to in the quotation above are for the years 1881, 1888 and 1890, for which years the water-way is roughly 6,600, 5,400 and 3,200 feet, the final width in 1917 being about 1,900 feet).

On page 120 the authors had given different methods for finding the maximum discharge in flood for Nullahs and Torrents. The speaker would like to draw their attention to Mr. Lacey's formula quoted above, from which, knowing the width of the water surface, it was possible, by inversion, to find the maximum flood discharge. The width of the water surface could usually be estimated from the flood marks, and it should not be difficult therefore to deduce the maximum discharge.

In case, however, the authors preferred to use a method based on the relation between rainfall and runoff, the best method for Punjab conditions was probably that given by Messrs. Khangar and Gulati, which had also been quoted by them.

He would also like to draw the author's attention to a formula given by Mr. C. C. Inglis, for maximum flood. The formula was given in Bombay P. W. D. Technical Paper No. 58. It was: Runoff $= \frac{7000\,A}{\sqrt{A+4}}$, where A is the catchment area in square miles. Originally, Mr. Inglis had given it in the form $Q=7{,}000\sqrt{A}$, in Bombay Technical Paper No. 30. He had since modified it to the form given first, to make it fit small as well as large catchments in India, provided the catchments were fan-shaped and not elongated.

On page 122 the authors had assumed that the bottom velocity equals $5V/6$, where V is the mean velocity. Some time back he had to analyse the velocity observations on some of the Punjab channels with a view to finding the depth at which the mean velocity on the vertical occurs. In these experiments velocities were taken on a vertical at ·1, ·2...... ·9 of depth. In order to determine the bed velocity, a reference was made to "The Hand-book of Professional Orders". The following formula was given there: $V_b = V_s + 1 - 2\sqrt{V_s}$ V_b and V_s being the bed and surface velocities. It was found that this formula gave a value for bed velocity appreciably lower than that obtained by extra-polating the plotted points against depth. He had referred to the Discharge Division for the authority behind this formula and had not received a reply so far.

It might, however, be pointed out that the formula quoted by me refers only to velocities on a vertical, and not to the entire section.

On page 123 the authors had stated that under equilibrium conditions the cross-section of the channel would tend to assume a form in which its perimeter was a ninimum for a given area, to achieve which the bed of channel after scour must assume a curved form.

Mr. Lacey had also assumed that the cross-section must have a minimum perimeter for a given area, when the channel was stable. He had, however, assumed that the shape of the channel must be a semi-ellipse.

The authors had assumed that the shape was given by a double parabola, for which the vertex was at the lowest point of the river bed. The two arms of the parabola were supposed to have unequal latera-recta.

This problem of finding the curve for which the perimeter was minimum for a given area, was about 2,000 years old, and had been first put forward by a Greek, Zenodorus. It was one of the famous problems in the Calculus of Variations, and the solution, as he had informed Mr. Lacey, was that the curve was an arc of a circle.

On page 124 the authors had used Kutter's formula. He would like to draw their attention to a recent book: 'Stream and Channel Flow' by Morgan (Chapman & Hall, Ltd.). In that, Kutter's formula had been compared in detail with that of Manning, and it was stated that except for large rivers with flat slopes, Manning's formula was superior. As Manning's formula lent itself to a very much simpler treatment, it should be possible to considerably reduce the treatment given by the authors on pages 124 and 125.

He heartily endorsed the author's statement on page 126 that it was uneconomical to provide a bridge for passing the highest floods, for such floods are rare and any damage was offset by savings on interest and maintenance.

At the bottom of page 126 the authors had given a summary of Mr. Lacey's conclusions in the first paper. In order to determine the depth of scour, in floods, he would like to draw their attention to para 9 of a note on "The Design of Hydraulic Structures resting on clayey Soils" by Rai Bahadur A. N. Khosla. This note was given at pages 169 to 173 of the Annual Report of the Central Board of Irrigation for 1939-40. Mr. Khosla had stated that the depth of scour could be worked out from the formula: $R=0.9^{\frac{2}{3}}$, where q was the discharge per foot run. Mr. Khosla also considered that the depth of scour was independent of the mean diameter of the bed particles.

On page 127 the authors had suggested that in large rivers the minimum width of stable waterway might be obtained from Mr. Lacey's formula, which had been quoted in para 2 of these comments. There was just one point in regard to this. On most of the Punjab Weirs it had been found that a certain factor of looseness had to be allowed for and that the actual width was some 20 to 80 per cent. greater than that given by the Lacey formula. Whether a similar factor should be applied for bridges was a point on which the authors might express an opinion. To a layman it would appear that it was easier to build the bridge to a slightly generous section, and then to close down a span or two, if the section was found to be too loose, rather than have a tight section, and run the risk of being out-flanked by the stream in a high flood.

Mr. G. R. Sawhny thanked the joint authors for writing this paper on a very important subject that still needed thorough investigation. From the way the authors had started about there being no contribution recently made for determining the waterway required under bridges, he had thought that they were going to show a new way to deal with the question, but his hopes had soon got shattered when no real solution was found to be forthcoming from the paper.

They had done useful work in putting together the various methods known for calculating river discharges but eventually had given no sound reason, for discarding the other well-known methods and for accepting the Lacey's Theory which as yet was in the melting pot. Their assertion about bed remaining stable at a certain stage of water level and their method of determining that stage, would solve a good deal of our trouble, if only this could be proved to be true.

The question arose how could the slope be assumed to remain constant in any stream?

The paper was another case of juggling with figures as a means to an end which the authors seemed to have had in their mind. They had been lucky that they had reconsidered the closing of only one bay out of three instead of the two as suggested by the Railway Board and thus had kept the water-way much more than might be required to pass the maximum discharge.

Author's assumption about the runoff from canal irrigated area was also incorrect.

"Our deeds are fetters that we forge ourselves." Authors were lucky in that they had made them on the loose side.

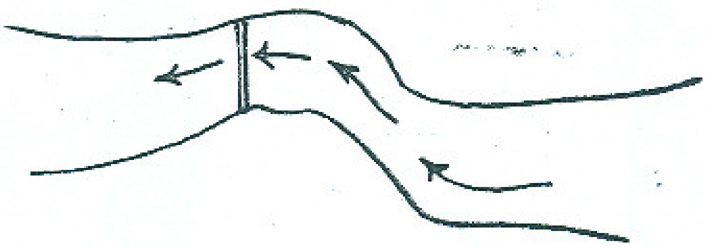
Mr. Haq Nawaz Khan Qamr remarked that before coming to the point of criticism he expressed his thanks to his friend Karnail Singh and his companion Gurdial Singh for bringing out this useful paper—so much so that he after a lapse of four years had felt the necessity of attending the Congress and coming to offer his comments for the first time since 1929.

In this paper the object had been to discuss and find out the economical waterway of bridges required for natural channels, but as that mainly depends on the correct determination of maximum discharge, he was afraid, the authors had completely failed to achieve their object because they had not been able to devise any new theory for determining the discharge. The theory of catchment basin was very old and gave only a rough idea as it was likely to be influenced by the soil and geological features of the catchment basin. It would certainly have been a great contribution to the Engineering profession if anyone could suggest more appropriate method of arriving at correct figures of such discharges but at least the authors had not—from what he saw in the paper he could say that it was simply a summary of various books of reference, so commonly available in head offices and that there was not at all any originality of thoughts or facts. In designing waterway of a bridge, determination of maximum discharge, its velocity in consideration with the soil were the main factors and on these pints the authors had given nothing new except rattling the old bones of Dicken, Dum and Waddle. As such, the speaker was afraid, they were only the compilers of this paper and not the authors. Any Head Draftsman working on checking of designs could write such papers every now and then, especially when being in Head Office they could find time, leisure and help of various books to write such things in justification of their existence. On page 1 it is said that Alexandra bridge had been reduced from 64 spans to 17—true, but who could say as to what were the circumstances then when this bridge was originally constructed. It was possible that at that time the condition of channel or the catchment basin had been such that it used to or was likely to carry much greater discharge than what was being assumed or foreseen now. The Railway bridges particularly connected with human life, were surely to be considered with broader vision and factor of safety. He admitted that engineers of the old time were rather overcautious in designing structures and, this overcautiousness would be found here and there and all over but that did not mean that they all had been fools. It was only due to the development of the younger generation and to the change of the conditions that more economical results were being achieved.

Mr. G. R. Nangea said that the fixation of waterway was of fundamental importance in the design of bridges. Leaving aside the mathematical calculations, all attempt to say something on this subject must necessarily consist in description of one or more bridge sites.

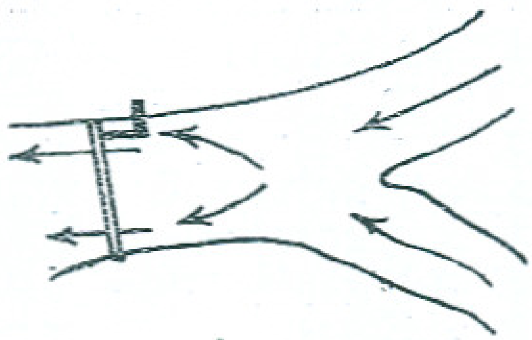
The particular example mentioned in the paper was the best known to the authors. Everybody would agree with them that they could train a stream to pass through a limited waterway provided the velocity of flow was not excessive and the cost of guide bunds was not prohibitive. The contraction in the waterway resulted in an increased velocity and the accompanying scour which must be provided against and its cost assessed before the final decision.

Where the banks were stable, an ideal bridge site with naturally contracted stream was afforded immediately below a bend. Here it usually formed a convenient neck, and if there is no further curvature in the course of the stream immediately downstream of the bridge, the current passes through it without any obstruction. Such an example is the Road Bridge over the Sohan at Dhok Pathan.



The ordinary width of that river was 1,400 to 1,800 feet but the bridge was only 880 feet in length. Here again, he must repeat what he had said while introducing his paper on the above bridge last year. The centrifugal force throws the main current on the outside of the bend and the resulting excessive scour had necessitated special design of foundations. The water level on that side was also higher and this required greater free-board.

Next he would discuss the hill streams. Attempt to considerably restrict the waterways had not always been successful. The Road Bridge at Chakki on Pathankote Baijnath Road was a case in point.



Originally, 20 spans of 40 feet arches had been built but later 10 more girder spans had to be added.

The bridge was situated half a mile below the junction of two big nullahs. Each stream, when in flood, tried to maintain its own direction. Thus the bed had scoured deep at both ends while the central spans had silted up. In 1933, the right side scoured so deep that five spans fell down and had to be rebuilt with extra deep foundations. The tendency of the stream to outflank this end had also to be counteracted by providing a spur. The standing right abutment had also been protected by a stone crate apron and a Bell's Bund.

Lastly, continuing his remarks the speaker invited the attention of members to the many streams where the natural width of the gorge was so insufficient that there had been enormous rise in water level. The site of the Harro Road Bridge at Chhoi on Campbellpur Basal Road was an instructive example of this. The old bridge (which was a beautiful piece of workmanship) was carried away simply because the decking was not high enough for an extraordinary flood like the one of the year 1929. Such cases called for very special care in deciding the decking level and where the height of the bridge rendered the cost excessive, submersible bridge was the only alternative. For the Railways, a high level bridge might be obligatory but for a Road Bridge it would be ideal to keep the decking at the level of the ordinary yearly high floods and to allow the maximum flood (which usually came once in many years) over it.

R. B. Wazir Chand Chopra remarked that in 1921 it fell to his lot to study the conditions and data of this 'Rohi Beas' crossing of the G. T. Road and to try to determine the orders and frequency of floods in the Nullah and to advise the Roads and Buildings Department about provision to be made there. This was at the instance of Mr. Cockburn, Superintending Engineer, 3rd Circle, P.W.D. His note of that time was fortunately extant in that Circle Office, which and the concerned file he had inspected by courtesy of their Vice-President, Mr. Kundan Lal, who now held the charge of that Circle. With the help of these papers he had written this discussion on Mr. Karnail Singh's paper.

The authors said that "no records are available to show as to why a waterway of three 100 ft. spans was provided" and that "it, appears that it was in conformity with the practice of ancient engineers to bridge the entire width of the flood nullahs". But fortunately, enough of record does, as a matter of fact, exist in the Railway Department to show, that at the time of the damage in 1894, this crossing consisted of a girder bridge of 3 spans, the central span being 34 ft. and the two end-spans 30 ft. each—roughly a waterway a little short of one span of 100 ft. The abutments and long wing walls were founded on wells, those of the east abutment being 31 ft. and those of the west abutment being 25 ft. below bed level. The east pier was founded on a well 18·4 ft. below bed, and the west pier on a large block of concrete which was laid on a mass of brickwork, evidently the old abutment of some former bridge, at 16 ft. below bed. At 23 hours on the night between 18th and 19th June, 1894, the bridge was wrecked by a flood caused by a very heavy rainfall in the catchment. The flood rose to within 7·6 in. of the rail level (including, of course, afflux caused by the bridge obstruction, the depth above floor being about 9 feet), and carried away the whole of the flooring and all the pitching of the abutments and wing walls. The east pier which rested on 18 ft. well foundation was washed away and the other pier stood. And, of course, deep scours took place and serious cracks appeared in certain wing and return walls. The traffic was blocked and through communication was restored at 14-45 hours on 21st June by putting a temporary sleeper pier. As a result of the lesson learnt from this disaster, the smaller three spans had been replaced by three spans of 100 ft. each; and the same waterway had been repeated when the railway line was doubled in 1911. This was a brief history of the bridge.

The 19th June, 1894, flood being the highest one, his first attempt had been to find out the rainfall in the catchment that caused this flood, and at the same time to find out if similar or heavier rainfall had taken place at other times, thus to try to determine the heaviest possible flood discharge. For a Railway bridge this last figure was enough, as best economy for the Railway is to provide for the highest discharge; but since his advice was to be rendered to the Roads and Buildings Department, he had tried to work out floods of the second and third orders also. This would cover the authors' idea explained in the first paragraph of Art. 4 of their paper, where for reasons of economy they thought of providing for less than the highest flood in certain cases for Railway bridges also.

In order to do all this, the speaker had ferreted out rainfall figures in the past and tried to work out flood discharges of different orders from first principles, by applying the results of his observations elsewhere, and making certain assumptions. These assumptions were clearly stated, so that any one differing from them might substitute the ones, his experience and sagacity led him to.

For this purpose he had dived into old rainfall records up to the year 1875, which he had known to be a year of very heavy rainfall. The canal rainfall stations nearest the catchment are Tibri and Sathiali near head reach, Athwal near middle reach, and Raya near tail at Grand Trunk Road. Record at Tibri started in 1876, Sathiali in 1893, Athwal in 1903, and Raya in August 1878. Prior to this, Gazettes were consulted back to 1875, and the two stations of Gurdaspur (near Tibri) and Tarntaran (comparatively nearest Raya) were taken just to have an idea of rainfall back up to 1875.

The rainfall corresponding to the flood of 19th June, 1894, was given in the table below:

Station.	*Rainfall in inches for 24 hours ending 8 a.m. of June, 1894.*			Remarks.
	17th	18th	19th	
Tibri ..	..	1·85	4·50	Athwal station was not yet started.
Sathiali ..	..	0·80	9·00	
Raya ..	1·71	1·61	6·72	

But rainfalls of the same or larger amount in 24 hours had occurred in August 1875, September 1877, August 1887, and September 1903 and the data for these was given below: (*See next page.*)

Station.	Rainfall.																Remarks.
	August, 1875						September, 1877			August, 1887			September, 1903				
	8	9	10	11	12	13	2	3	4	12	13	14	10	11	12	13	
Gurdaspur, Tibri ..	0·6	2·6	*5·3*	0·2	*9·0*	0·2	0·7	*9·6*	0·4	..	*6·7*	1·0	1·4	4·3	*7·5*	1·0	Tibri and Gurdaspur may be taken as corresponding stations also for want of better proximity Raya and Tarntaran.
Sathiali ..	..	..	..	..	..	..	..	..	..	..	..	..	0·7	2·3	*9·2*	..	
Athwal ..	..	..	..	..	..	..	..	..	..	..	..	..	1·0	2·5	*7·1*	..	
Raya, Tarntaran ..	1·0	1·8	*9·3*	0·6	*7·8*	1·7	0·7	*4·3*	..	..	*9·5*	2·0	..	2·9	*9·8*	..	
			(1)		*(2)*			*(3)*			*(4)*				*(5)*		
	Canal Stations not yet established.						Only Tibri Canal station established.			Only Tibri and Raya Canal Station established.			All Canal stations established.				

Rainfalls (1) and (3) might be taken of the same degree as 1894, while (2), (4) and (5) were decidedly heavier, especially August, 1875, and

12th September, 1903, which had also been preceded by wet periods. The effect of 1875 floods (1) and (2) was not known, but must have been awful. September, 1877, flood (3) evidently did not cause damage, as although rainfall had been high at the top, it had been low at the bottom of the catchment. August, 1887, flood (4) was really worse than 1894 and the bridge must have escaped by sheer luck, as this flood did wreck the Naraingarh Bridge between Jandiala and Butari (about 12 miles from Beas station) necessitating lengthening it by 120 ft. The 1903 flood (5) would have played havoc with the original bridge, but fortunately the larger bridge had been built. Since three-100 ft.-span bridge had seen this flood, it could be considered as amply safe for all time, and for this reason perhaps its waterway was considered too liberal and was intended to be cut down.

Thus, this catchment, under worse conditions, was subject to heavy rainfall of 9 inches in a day preceded and followed by wet weather. It might be assumed then, that such conditions were probable in 1875, 1877, 1887, 1894 and 1903—*viz.* at intervals of 2, 10, 7 and 9 years, or ignoring the first, say at an interval of 9 years. The catchment area of this drainage was 73·86 or roughly 75 square miles. On a comparatively smaller catchment with 4 ins. of rainfall, in a less sandy but fairly well-cultivated (not irrigated) tract in Sialkot District, he had found that the maximum discharge represented 15 per cent. of the total rainfall. This did not mean that 85 per cent. of the rainfall had been absorbed but that the rate of flow had at no time exceeded 15 per cent. In the present case, the factors were :

(1) Country more sandy, therefore, percentage less.

(2) Catchment area larger therefore, percentage less.

(3) Fall in country much the same, percentage same.

(4) Both fairly well cultivated, but not irrigated (irrigation did not exist in this catchment in 1894), therefore, percentage same.

(5) Rainfall about double in quantity, percentage higher.

(6) Intensity evidently much greater, percentage higher.

On the whole, he assumed that percentage to be 20 in this case of heavy rainfall of 9 inches of this nature.

If the rain had fallen uniformly in 24 hours, and taking 20 per cent. in the case of this steady rainfall also as a precaution the discharge would work out to

$$\frac{9\times 75\times 5280\times 5280\times 20}{12\times 24\times 60\times 60\times 100}=3630 \text{ cusecs.}$$

but when heavy floods came, the rainfall was often such that major portion fell in comparatively short time and balance was spread over the rest of the day. He could only make certain assumption from experience and guess work, and he for one would assume that $\frac{3}{4}$ of this rainfall would under really heavy conditions occur say in 8 hours. The practical maximum flood, which under Barani conditions may be expected, would then be

$$\frac{\frac{3}{4}\times 9\times 75\times 5280\times 5280\times 20}{12\times 8\times 60\times 60\times 100}=8{,}167 \text{ or say } 8{,}000 \text{ cusecs.}$$

But since the country was now irrigated mostly, and therefore highly cultivated, the discharge now would be much less, and the runoff percentage might be just half resulting in 4,000 cusecs, under heaviest rainfall conditions. He did not at all agree with what the authors said on page 129, that the effect of irrigation should be to increase the runoff. On the contrary, irrigated, and therefore intensely cultivated tracts, yield much less runoff than Barani or Banjar areas due to ploughing and high and numerous field ridges.

Other periods of heavy rainfalls that might count—namely such as when any of the stations had recorded over 5 inches rainfall, or when good general rainfall had taken place, had also been collected. They were 18 in number and were given in the table attached at the end. Of these Nos. (1), (2) (*ii*) and (16) might be ignored as regards flood effects, as having a good rainfall of over 5 inches at the top and nothing or very little lower down. The value of Nos. $\frac{(3),\ (7) \text{ and } (13)}{(a)}$ where at the lower end the rainfall was really heavy with very little at the top, and of Nos. $\frac{(9),\ (12) \text{ and } (17)}{(b)}$ where rainfall at the top was really heavy and fell off considerably and sometimes to nothing towards the bottom, it amounted to a rainfall of $3\frac{1}{2}$ inches to $4\frac{1}{2}$ inches. The tendency of No. (10) was such that it could have been like No. (18), so he would assume, that there was a possibility of say a uniform rainfall of 6 inches over the catchment area once in 17 years, or say 20 years. The remaining six, namely Nos. $\frac{(4),\ (5),\ (6),\ (8) \text{ and } (15)}{(c)}$ meant an average of 4 inches to $4\frac{1}{2}$ inches rainfall on the catchment area. Taking groups (*a*), (*b*) and (*c*) together, and putting them in their chronological order, the intervals were 2, 1, 12, 3, 1, 2, 6 and 6 years. Twelve years appeared exceptional, while 5 and 6 seemed also rare. The normal interval was 1 to 3 years, and we might take 2 years.

The above analysis showed that there were three orders of heavy rainfall and consequent floods in this nullah :

1st order : Due to a heavy rainfall of 9 inches in a day, say once in 10 years.

2nd order : Due to a heavy rainfall of 6 inches in a day, say once in 20 years.

3rd order : Due to a rainfall of $4\frac{1}{2}$ inches, every other year.

The discharge from the 1st order had been worked out in para 8 above, *viz.* 8,000 cusecs. For the other two orders he would make the same assumption of $\frac{3}{4}$ inches of rainfall in 8 hours, and would take 17 and 15 as percentages of runoff. The discharges under Barani conditions would then be

$$\text{for } 2nd \text{ order } 8{,}167 \times \frac{6}{9} \times \frac{17}{20} = 4{,}628 \text{ or say } 4{,}500 \text{ cusecs}$$

$$\text{and for } 3rd \text{ order } 8{,}167 \times \frac{4\frac{1}{2}}{9} \times \frac{15}{20} \times 3{,}063 \text{ or say } 3{,}000 \text{ cusecs.}$$

Under the present irrigated conditions the discharges might be assumed half of the above, namely 4,000, 2,250 and 1,500 cusecs respectively.

The next thing to determine was the depth and velocity, with which each of the above figures of discharge would flow in the nullah. The nullah had a bed-width of 600 ft. and the slope was not less than 15 ft. in 10,000 ft. or 1 in 667. With these data, the velocities and depths were worked out by trial and error, and were given in the table below :

S. No.	Condition.	Discharge cusecs.	Depth ft.	Velocity f/s.	Probable interval.
1	2	3	4	5	6
	1st order :				
1.	Barani ..	8,000	3·2	4·3	10
2.	Irrigated ..	4,000	2·2	3·0	
	2nd order :				
3.	Barani ..	4,500	2·3	3·3	20
4.	Irrigated ..	2,250	1·6	2·5	
	3rd order :				
5.	Barani ..	3,000	1·9	2·9	2
6.	Irrigated ..	1,500	1·2	2·1	

RAINFALL.

Station.	1870 July. 1st period 13	14	15	16	17	2nd period. 28	29	30	31	August 1878. 21	22	23	July 1880. 1
Head Reach. Gurdaspur ..	..	..	*5·3*	·2	..	..	2·2	*5·2*	..	..	..	..	..
Head Reach. Tibri ..	..	..	..	..	..	..	..	..	..	..	*0·4*	..	3·7
Head Reach. Sathiali ..	..	..	..	..	..	..	..	..	..	..	..	..	..
Mid Reach. Athwal ..	..	..	..	..	..	..	..	..	..	..	..	..	..
Bottom Reach. Raya ..	..	..	..	..	..	..	..	..	..	..	*7·0*	..	5·2
Bottom Reach. Tarntaran ..	..	..	..	..	..	..	..	..	..	..	..	..	..
			(1)					*(2)*			*(3)*		*(4)*
Average ..											*3·7*		*4·5*

Station.	July 1881. 13	14	15	1893 September. 16	17	18	19	20	1896 June. 24	25	26	1897 August. 20	21	22	23
Head Reach. Gurdaspur ..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
Head Reach. Tibri ..	1·7	*6·3*	1·0	..	0·6	3·9	*3·2*	2·0	·1	*1·7*	..	·9	*6·6*	1·9	1·2
							3·4			*·85*			*4·95*		
Head Reach. Sathiali ..	..	..	..	..	..	1·6	*3·5*	2·6	..	..	2·0	..	*3·3*	1·5	·5
Mid Reach. Athwal ..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
Bottom Reach. Raya ..	1·2	*3·0*	·7	·9	·9	·4	*4·5*	1·2	..	*6·4*	..	..	*2·1*	·2	·9
Bottom Reach. Tarntaran ..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
		(5)					*(6)*			*(7)*			*(8)*		
Average ..		*4·7*				*3·9*				*3·6*			*3·5*		

Station.	July and August. 31	1	2	1900 August. 6	7	8	9	10	11	1900 September. 10	11	12	1900 Jul 25	26
Head Reach. Gurdaspur ..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
Head Reach. Tibri ..	2·0	*6·3*	·4	·2	·5	*5·3*	1·3	·5	·9	..	*5·7*	1·4	..	*8·2*
		6·5												*7·75*
Head Reach. Sathiali ..	·5	*6·7*	·4	2·2	1·7	*3·5*	1·4	0·3	1·6	..	*1·6*	·8	..	*7·3*
Mid Reach. Athwal ..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
Bottom Reach. Raya ..	1·0	..	..	1·0	..	*5·5*	·3	..	..	..	*1·3*	·7	..	·6
Bottom Reach. Tarntaran ..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
		(9)				*(10)*					*(11)*			*(12)*
Average ..		*3·2*				..					..			*4·2*

Note:—In striking average where Athwal (mid reach) is not recorded the average of Raya and mean of Tibri and Sathiali (the two head reach stations) has been taken—namely in case of Nos. (6), (7), (8), (9), (12) and (13).

RAINFALL.

July 1881.		1893 September.					1896 June.			1897 August.				July and August.		
14	15	16	17	18	19	20	24	25	26	20	21	22	23	31	1	2
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
6·3	1·0	..	0·6	3·9	*3·2*	2·0	·1	*1·7*	..	·9	*6·6*	1·9	1·2	2·0	*6·3*	·4
					3·4			*·35*			*4·95*				*6·5*	
..	..	..	..	1·6	*3·5*	2·0	..	..	2·0	..	*3·3*	1·5	·5	·5	*5·7*	·4
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
3·0	·7	·9	·9	·4	*4·5*	1·2	..	*6·4*	..	..	*2·1*	·2	·9	1·0	..	..
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
(5)					(6)			(7)			(8)				(9)	
4·7		*3·9*					*3·6*			*3·5*				*3·2*		

1900									1901							1903 July.		
August.						September.			July.				August.					
6	7	8	9	10	11	10	11	12	25	26	27	6	7	8	9	22	23	24
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
·2	·5	5·3	1·3	·5	·9	..	*5·7*	1·4	..	*8·2*	·4	·9	·4	*·9*	·2	·8	*4·3*	1·0
										7·75				*·75*				
2·2	1·7	3·5	1·4	0·3	1·6	..	*1·6*	·8	..	7·3	1·5	·8	0·8	*·6*	·2	·2	*3·1*	·1
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	1·1	*6·4*	1·1
1·0	..	5·5	·3	..	..	..	*1·3*	·7	..	·6	·6	·6	1·2	*6·6*	..	·2	*2·9*	·2
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
		(10)					(11)			(12)				(13)			(14)	
..						..			*4·2*									

1909 August and September.									1912 July and August.			1914 July.			October 1917.	
26	27	28	29	30	31	1	2	3	31	1	2	3	4	5	25	26
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
1·4	1·5	·6	1·3	·9	*5·9*	·7	1·4	2·5	..	*3·4*	·9	2·4	*5·7*	·6	·3	*5·6*
2·8	·5	·8	2·9	·6	*2·8*	·7	1·3	3·2	1·7	*5·6*	2·4	2·8	*4·2*	·5	·3	*5·5*
3·6	·3	·3	1·0	·1	·7	1·0	1·7	3·4	·1	*1·7*	1·6	·3	*3·9*	·4	·6	*6·9*
·6	·5	·7	1·0	·5	*2·2*	·6	2·6	2·5	·5	·5	3·1	..	*1·6*	·8	·5	*5·8*
..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
					(15)					(16)			(17)			(18)
2·9									..			*4·4*			..	

...aya and mean of Tibri and Sathiali (the two head reach stations) has been taken—namely in case of Nos. (6), (7), (8), (9), (12) and (13).

In considering the design for a bridge, the figures of depth led to the form of a long and low bridge, rather than a high and narrow one. Also since floods of the 2nd order were of lower frequency than those of the first order, they did not need to be considered. To the Roads and Building Department he had advised to build work for 1,500 cusecs in the shape of a causeway or a submersible bridge for a suitable higher discharge allowing still higher floods, which would be less frequent, to pass by overflow. As regards waterway to be kept in the case of the Railway bridge, it could be seen that practically one span of 100 ft. with foundations from 18 to 31 ft. deep, gave way. But when the depth of foundation as now was about 50 ft., one span of 100 ft. should stand all right. But since waterway was already available, it had on the whole been wise that two spans were being retained instead of one. And two spans should always be safe. Three spans were really an over-liberal provision. By Lacey's formula also it worked out to $2{\cdot}67\ (4000)^{0{\cdot}5} = 169$ ft., which meant two spans in practice. 200 ft. was more useful otherwise also, as the nullah was not a river, but was really a cutting in the naturally high bank of the river Beas (and receiving no spill from it), and scour would not be of the same nature as in the case of a river with deep bed of sand made of the detritus from the hills.

While replying to the discussion on the paper S. KARNAIL SINGH stated that the authors were grateful to R. B. Wazir Chand whose comments on the second part of the paper, referring particularly to the Rohi-Beas bridge on North Western Railway near Beas Railway Station, had greatly added to the usefulness of the paper. R. B. Wazir Chand Chopra contributed the authentic history of this bridge, which the authors had not been able to get at in the course of their investigations. The note by Rai Bahadur was constructive and no replies were called for. It was a matter of pleasure to the authors that the conclusions arrived at by Rai Bahadur Chopra, in the light at his personal experience nearly 20 years ago, agreed with those arrived in the paper regarding the required waterway under this bridge. Dr. Malhotra's contribution, in the form of discussion, was very much appreciated, as his observations were constructive and appreciative of the work done in the paper. It was, however, pointed out that Mr. Lacey's paper on the subject was read with interest by the authors and they had summarised his conclusions in the paper, a good portion of which was really based on Lacey's theory. The authors did not, however, agree with Dr. Malhotra that the discharge of a catchment can be found by an inversion of Lacey's formula. As the authors understood it, the width of waterway was dependent on an semi-elliptical cross-section of the channel, the velocity in which was equal to critical velocity. And if water was spread over a larger width, the velocity, and consequently the scour depth, was reduced and, therefore, the actual discharge with the application of the formula proposed by Dr. Malhotra was not workable. The conclusion was also borne out by the example of the contraction of waterway on the

Alexandra Bridge. The authors agreed with Dr. Malhotra as regards the usefulness of Messrs. Khangar and Gulati's method, but had to point out that its limitations regarding the requirement of data, etc., were considered to be far too many for general application. In para 6 Dr. Malhotra suggested the substitution of Manning's formula for Kutter's formula. The two formulas were practically identical and the authors preferred to leave the application of one or the other to the discretion of the Engineers. Rai Bahadur Khosla's formula for scour, quoted by Dr. Malhotra, was actually a derivation from Lacey's formula assuming the value of 'f' as unity. The derivation had been given by Rai Bahadur Khosla on page 131 of his paper on "Design of Weirs and Permeable Foundations," published by the Central Board of Irrigation. The comparison of the width of waterway between the bridges and over the weirs was hardly compatible as the governing conditions in the two cases were vastly different.

Mr. Sawhney's attention was invited to the introduction of the paper, in which it had been made clear that the authors had attempted to compile the relevant information as already available in different publications. It was not understood what theories, other than that of Lacey, did Mr. Sawhney refer to. The authors were only aware of Sir Francis Spring's "Thumb Rule" suggesting the bridging of about a third of the whole waterway. There was a lucky coincidence that the results obtained with the application of Lacey's formula and according to Spring's "Thumb Rule" were, more or less, the same. It was suggested that Mr. Sawhney will do well to go through Sir Francis Spring's paper to know the correct information about the slopes of rivers in the plains. Incidentally, it was not the Railway Board who had suggested the contraction of waterway but the Divisional Superintendent, who was an officer subordinate to the General Manager on the Indian Railways.

In conclusion, S. Karnail Singh remarked that the authors' were grateful to Mr. Haq Nawaz Khan for entertaining the members of the Congress, and it was sincerely hoped by the authors that the ignorance betrayed in his remarks as regards the waterway on the Alexandra Bridge was a deliberate attempt for amusement and that his criticism was not meant to be taken as the considered remarks of an Engineer.