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Paper No. 216

"FLOOD DISCHARGE ESTIMATION FOR SMALL CATCHMENTS"

By

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CONTENTS

SYNOPSIS

The Paper describes a method of estimating flood discharge from small catchments in Andhra Pradesh by making use of the data available from rain gauges. The rain gauge data has been analysed and from that the hourly intensity of rainfall that could be expected by an analogy of the rainfall characteristics collected from the two automatic recording rain gauge stations at Begumpet and Visakhapatnam has been attempted.

The Paper also describes the use of this hourly intensity to get the flood discharge.

Illustrative examples to show the application of the above data to actual problems met with in Andhra Pradesh have been included.

A map of Andhra Pradesh with zones of equal hourly intensity has been prepared to select suitable values for evaluating flood discharge in different localities.

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1. INTRODUCTION

1.1. It is well known that basic data like rainfall figures and field characteristics of catchments is not easily available to estimate the correct runoff for a proper design of the cross drainage works. No doubt the problem has been studied by many engineers and each has evolved a formula of his own for a particular locality, but the basic data in regard to factors that affect the flow from a particular catchment is not usually available to the designer to analyse each case on its merits.

1.2. As most of these fomulae tend to generalise all cases, they are not fully satisfactory. Individual catchments, especially the small ones, have a strong influence on the amount of runoff according to their peculiarity, whereas a ready-made formula takes into account only the average characteristics. It is, therefore, necessary that, in addition to the formulae, flood discharge should be arrived at from an individual study of each catchment, taking into account the basic factors that influence the flood.

1.3. The basic factors to be assessed for obtaining flood discharge—

$$Q = KAi \qquad \ldots \qquad (1)$$

for the catchment under study are:

(1) The area of the catchment (A);

(2) The average effective intensity of rainfall on that catchment (i);

(3) The percentage that could be expected from that area called the coefficient of discharge (K).

These depend, of course, upon the locality, nature of catchment, its initial condition, shape and other factors.

1.4. The object of this Paper is to collect and interpret such data as is available for estimating flood discharge at any point of a stream in Andhra Pradesh by making use of the above formula.

1.5. Flood estimation for big projects is usually done with special staff and with considerable effort. Usually, it is the small bridge or a culvert where so much effort is not bestowed for arriving at a reliable runoff. The scope of this Paper has, therefore, been confined to the study of small catchments.

1.6. It is difficult to define precisely a small catchment. It may be taken as "that area which may be expected to get covered under one cloudburst lasting sufficiently long to be effective on the catchment".

1.7. An idea of the extent of the cloud coverage that may be expected can be formed from a perusal of the isohyetal maps given in Plate II (a,b,c,d) for different areas in Andhra Pradesh. This does not mean that the rain fell all over that area at the same time. It only represents the rain that fell in 24 hours over that area. As will be seen in subsequent Chapters, this covering alone is not the criterion; it is also essential to know whether it persists long enough to suit the time factor of the particular catchment. This requires a study of the duration of rain over wider areas and also a study of the probability of its simultaneous occurrence and the periods.

1.8. When it is raining over an area A', whether it is also raining at the same time over a wider area of the same catchment A''. This comes under the realm of probability called the "Combinatory analysis". The probabilities of occurrence of a set of n independent events are $p_1, p_2, p_3 \ldots\ldots\ldots p_n$, the possibility that all the sets of events will occur is:

$$p = p_1 \; p_2 \; \ldots\ldots\ldots \; p_n$$

Thus, let there be only two events whose probability of success is $p_1 = \dfrac{m_1}{n_1}$ and $p_2 = \dfrac{m_2}{n_2}$

$$\text{then } p = \frac{m_1 \, m_2}{n_1 \cdot n_2}$$
$$= p_1 \; p_2$$

1.9. As an example, if it is required to find the probability of occurrence of, say, "four hours" of continuous rainfall in two areas of which one area receives 16 hours of rainfall and a larger area in the same locality 12 hours of rainfall, then

(i) the probability of rain of 4 hours duration taking place in the first area A' is 16/4, i.e., 1 in 4;

(ii) the probability of raining 4 hours continuously in the the area A'' with 12 hours rainfall = 12/4, i.e., 1 in 3. Hence the probability of success in both the areas = 1 in 4×3 = 1 in 12.

Thus the probability is there but the frequency of raining on the whole catchment for the required duration to produce a maximum flood depends upon the size of the catchment and its time factor (explained later).

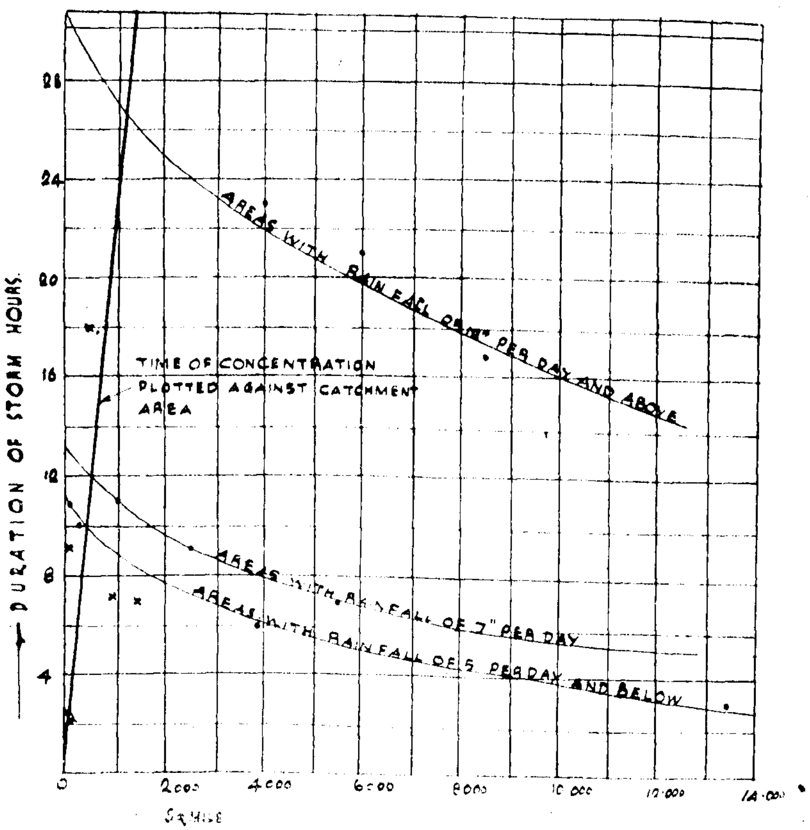


Fig. 1

1.10. To get a general idea of the size of the catchment to be called a "small catchment" for the purpose of this Paper, the duration of storms over areas has been plotted taking the average rainfall for the whole period as about half inch intensity per hour and making use of the isohyetals for different areas. The time factor calculated for different sizes of catchment has been superimposed over the above graph, Fig. 1, to give the area considered suitable for the application of the data collected. Of course, this will give only a rough idea, as the time factor for a catchment, as will be seen later, is not a function of the size of the catchment alone.

1.11. From the above it could be roughly concluded that the following areas might be considered as 'small' to apply the data collected and interpreted in the following pages.

	Maximum size of catchment that could be taken to cover 'small catchment'
1. For areas with 12 in. of rainfall per day and above.	800 to 1,000 sq. miles depending on the steepness of the catchment and nature of the soil.
2. For areas with 7 in. of rainfall per day.	400 sq. miles.
3. For areas with 5 in. of rainfall per day.	300 sq. miles.

1.12. If the area is larger than the areas contemplated above, additional factors will have to be taken into account. It may be that the catchment is under the influence of two or more storms at different places and at different times. Their combination is not a straight phenomenon. If the direction of the storm is in the direction of flow, it is likely that the flood will build up early as the upstream waters will have been on their way to join the downstream flood and create a peak flood.

2. PRESENT METHODS OF FLOOD ESTIMATION AND THEIR LIMITATIONS

2.1. The present procedure followed in Andhra Pradesh for arriving at a flood discharge for a small bridge or a culvert which does not warrant a detailed investigation is to calculate it by —

(i) Modified Dickens' formula

(ii) Ryves' formula

(iii) Area velocity method

Each of these methods gives result which is different from the other and it is a problem to fix a suitable figure from judgment. If a particular method out of the three is taken for purposes of design, it may result in some bridges getting under or over-designed. This point could be illustrated from examples that have been selected of streams in two different areas in Telengana portion of Andhra Pradesh. The two districts selected have different terrain and rainfall characteristics. The coefficients for Dickens' and Ryves' formulae have been adopted according to the size of the catchment and as fixed by the local P. W. D. Ryves' formula is generally used in Telengana for the design of irrigation works and it is not expected to give the maximum flood. Dickens' formula is expected to give the maximum runoff and needs only to be considered here. The districts chosen are Adilabad and Mehabubnagar which are to the north and south of Hyderabad respectively. Rainfall is heavier in Adilabad as compared to Mehabubnagar.

2.2. (1) **Streams in Adilabad District**

(a) Rainfall 37.02 in. average per year.

(b) Steeper catchment.

TABLE 1

No.	Area of catchment (sq. miles)	Discharge by Ryves' formula $=600\ A^{\frac{2}{3}}$ (cusecs)	Discharge by Dickens' formula $=1200\ A^{\frac{3}{4}}$ (cusecs)	Discharge by Area Velocity method* (cusecs)	Remarks
1.	16	4,640	8,000	1,400	Area velocity method gives the highest discharge in this district
2.	26	6,380	11,500	12,100	
3.	177	23,200	41,500	58,000	
4.	37	8,156	15,200	15,200	

* Surplus course of a tank.

(2) **Streams in Mehabubnagar District**

(a) Rainfall 22.95 in. average per year

(b) Flat catchment

TABLE 2

No.	Area of catchment (sq. miles)	Discharge by Ryves' formula $=600\ A^{\frac{2}{3}}$ (cusecs)	Discharge by Dickens' formula $=1200\ A^{\frac{3}{4}}$ (cusecs)	Discharge by Area Velocity method (cusecs)	Remarks
1.	1.5	786	1,727	1,314	Dickens' formula gives the maximum discharge in this district
2.	1.15	659	1,333	1,020	
3.	17.4	4,002	8,500	2,760*	
4.	1.46	775	1,600	1,685@	
5.	2.78	1,190	2,580	4,500@	

* Surplus course of a tank.

@ Immediately below a tank.

2.3. The bridges in Adilabad District will be designed according to discharge given by "Area Velocity" method as it gives the maximum runoff and the bridges in Mehabubnagar District will be designed according to the Dicken's formula as it gives the greater discharge. It is likely that the bridges in Mehabubnagar may be designed with the necessary or excessive waterway and the bridges in Adilabad district may be getting under-designed. The Area Velocity method has no proper check while the Dickens' discharge is only a function of the area with no relation to the special variations of the natural factors in these two districts.

2.4. These examples also show (*vide* items 4 and 5 of Table 2) that special features have no effect on the Dickens' discharge figures.

2.5. It may be concluded that unless the runoff given by the Area Velocity method is checked by another method, which also takes into account several local factors, it is not desirable to fix the discharge from this data alone.

2.6. Even though the runoff obtained from the Area Velocity method should be sufficient for computing the discharge, it is seen from experience that it is not fully reliable for the following reasons:

(1) The full flood section is not usually obtained.

(2) No proper measurement of the flood slope is made due to wrong information in regard to correct High Flood Line, which is usually ascertained by local enquires.

The extent of the errors that are likely to be made by these two methods could be seen from the following examples:

Error due to non-availability of the area of full flood section

2.7. Usually the area of the full flood section of the stream is not taken during a high flood, but is taken after the floods have subsided. This will naturally be a silted section of the river. This point is completely ignored and the calculations are made out as if the final section after the flood subsides is the same as during the flood with the high flood line recorded in the former case. Consequently, the cross section not conforming to the actual section will not give the correct discharge.

2.8. Coefficients assumed in the Manning's or Kutter's formula for calculating velocity, which depend upon discretion and are more correctly applicable to controlled channels, are used to arrive at the velocity of flow.

2.9. The extent of the day to day variations of the cross section of a river during a rising and falling flood is given in Plate III. The cross sections given there were plotted after gauging a river in actual floods.

2.10. The possible error that is likely if the flood discharge is calculated on the 12th day as compared to the peak flood on the 5th day, is worked out below by superimposing the maximum H.F.L. on the cross section of the 12th day, Fig. 2.

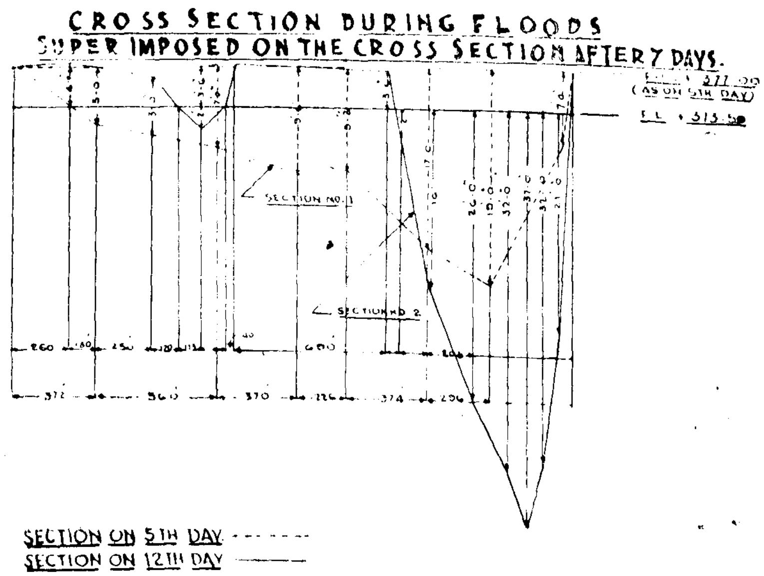


Fig. 2

2.11. Considering the flood slope and coefficient of rugosity to be the same on both these days (as it is the same river and flood), the velocity V in the Manning's fomula $V = \frac{1.485}{n} R^{\frac{2}{3}} . S^{\frac{3}{2}}$. will be $V = K(R)^{2/3}$, then if Q is the discharge on the 5th day and Q' is the discharge on the 12th day and R, A and R', A' the H.M.D. and cross sectional areas respectively for these days,

$$\frac{Q}{Q'} = \frac{K.A.}{K.A.'} \times \frac{R^{2/3}}{R'^{2/3}} = \frac{A}{A'}\left(\frac{R}{R'}\right)^{2/3} \qquad ...(2)$$

TABLE 3

Flood section of the fifth day (maximum flood)

S. No.	Ordinate	Mean Ordinate (x)	Distance (y)	Area (xy)	Wetted perimeter $\sqrt{(x^2 + y^2)}$
1	2	3	4	5	6
		Ft	Ft		
1.	...	...	...	...	...
2.	5.00	2.5	372	930.0	372.00
3.	7.00	6.00	560	3,360.0	560.00
4.	9.50	8.25	370	3,052.0	370.1
5.	9.16	9.33	226	2,109.3	226.00
6.	17.00	13.30	374	4,974.2	374.04
7.	19.00	18.00	296	5,328.0	296.00
8.	10.00	14.50	245	3,552.5	245.06
9.	7.00	8.50	80	680.00	80.07
10.	0.00	3.50	30	105.00	30.80
			Total	24,091.00	2,554.1 ft

$$\text{Hydraulic mean depth} = \frac{A}{P} = R = \frac{24091.0}{2554.1}$$

$$= 9.5 \text{ ft}$$

Inserting these values in equation (2)—

$$\frac{Q}{Q'} = \frac{24091}{21987}\left(\frac{9.5}{8.7}\right)^{2/3} \times 100 = 115 \text{ per cent approximately.}$$

2.12. Thus it would be seen that in this case the error is about 15 per cent under-estimation of the flood after seven days of the recession of the flood. This may be more if the section is taken on a subsequent day or on a dry bed.

2.13. The record of the full flood section is seldom available unless special efforts are made to gauge the rivers during the

TABLE 4

Flood section of the twelfth day

S. No.	Ordinate	Mean Ordinate (x)	Distance (y)	Area (xy)	Wetted perimeter $\sqrt{(x^2+y^2)}$
1	2	3	4	5	6
1.	0.0	0.00	372	0.0	372
2.	0.50	0.25	250	62.50	250
3.	5.50	3.00	233	699.00	234
4.	3.50	4.50	113	508.00	114
5.	0.00	1.75	40	70.00	41
6.	0.00	0.00	690	0.00	690
7.	19.50	9.75	210	2,050.00	211
8.	29.50	24.50	205	5,020.00	217
9.	40.50	25.00	245	8,600.00	247
10.	24.50	32.50	140	4,550.00	143
	—	12.25	35	430.00	37
			Total	21,987.00	2,556

$$\text{Hydraulic mean depth} = \frac{A'}{P'} = R' = \frac{21987}{2556}$$

$$= 8.7 \text{ ft}$$

floods. If this work is taken up by special staff engaged on bridge work, as no work is likely to be in progress during that period, it will help in selecting suitable coefficients in Dickens' formula as well as in the Manning's and Kutter's formulae for natural channels.

Wrong assessment of the high flood level.

2.14. The flood marks are usually ascertained by local enquiry and are not always reliable as could be seen from the following example selected from an estimate that came for scrutiny. Here the high flood mark at different sections of the same stream is given. Even though the width and cross section are varying it would be seen that the *flood slope* has been constant, which cannot be the case unless the cross section has been uniform throughout.

TABLE 5

Stream: Geedimonivampu

Catchment area, 2 sq. miles

S. No.	Section	Area in sq. ft.	Wetted perimeter	Slope	Velocity in ft per second	Discharge (cusecs)
1.	5 chains U/s	738	447 ft	0.0075	5.95	4,390
2.	12 ,, ,,	291	182 ,,	0.0075	5.86	1,710
3.	At crossing	750	340 ,,	0.0075	7.33	5,560
4.	10 chains D/s	221	173 ,,	0.0075	5.09	1,165
5.	25 ,, ,,	258	120 ,,	0.0075	7.15	1,842

The discharge given at various sections varies widely though no stream of importance joins this stream in the length under consideration. It would thus be seen that, unless there is a rational method, it is very difficult to fix a suitable runoff for the stream at any point.

2.15. Realising the unreliability of the flood discharge given by the Dickens' formula so far by using the prevailing constants, Sarvashri P.R. Ahuja and Shenoy[7] undertook a study of the rivers in Andhra Pradesh to fix the value of C in the Dickens' equation. They studied thirty three streams and rainfall of the area and correlated their computed discharge to the size of catchment areas and the value of C.

They observed that the value of C is higher for smaller catchments and is reduced as the size of the catchment increases. This is to be expected as the effective intensity of rain on smaller areas is likely to be higher. They also noticed that this was not the only factor affecting the discharge and found many points not falling on the straight line drawn to compute discharge against area, Fig. 3. This is explained as due to the peculiarities of the catchment like its shape, slope, location, and other local

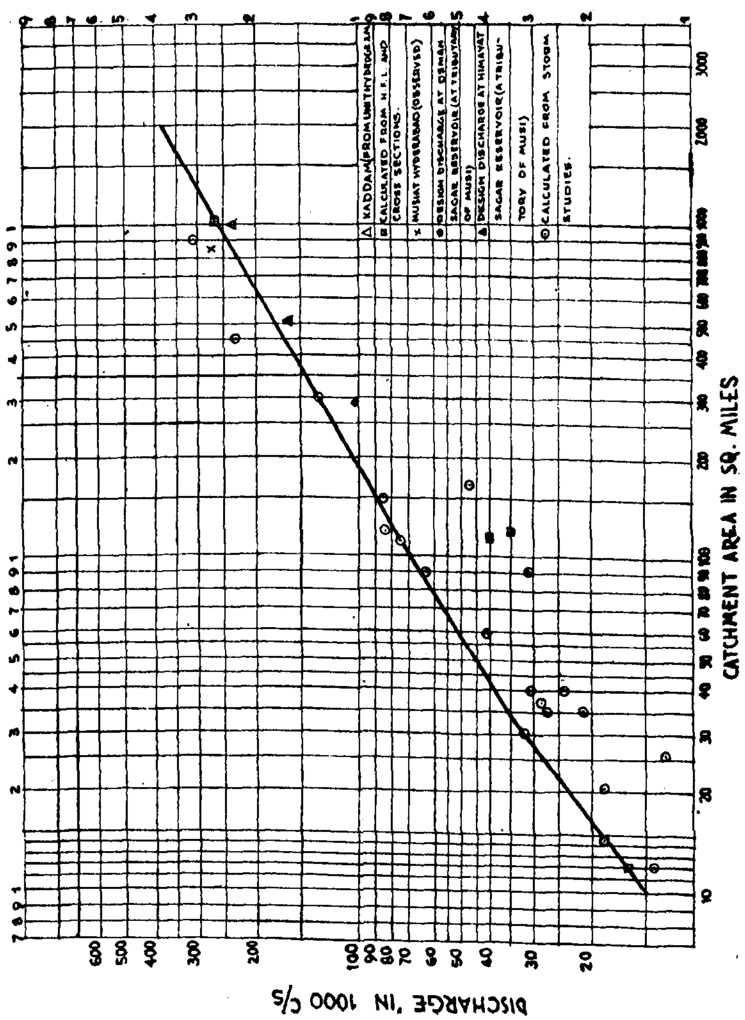


Fig. 3

features. They have also fixed broadly the following revised values for C in the Dickens' formula for future adoption:

TABLE 6

Size of catchment in sq. miles	Recommended value of C in Dickens' formula
Less than 30	2,400
30 – 100	2,200
100 – 300	1,900
300 – 1000	1,500

2.16. The basic concept of the method used in this Paper is that the maximum rate of runoff from a small drainage basin occurs when the entire basin is contributing, and that this rate of runoff equals a percentage K of the average rate of rainfall. In equation form:

$$Q = KAi$$

where Q is the rate of runoff[6] in acre inches per hour or cubic feet per second (1 acre inch per hour = 1.008 cubic feet per second, which may be taken as unity for all practical purposes), K is the ratio of peak runoff to average rainfall, i is the average rainfall intensity in inches per hour and A is the drainage area in acres. Implicit in the determination of i and in the stipulation "when the entire basin is contributing" is the determination of what is usually called the time of concentration, T_c. As visualised by earlier investigators T_c is the time of travel of a water particle from the most remote point to the outlet of the drainage basin. Although this concept of T_c is an oversimplification—assuming it for the moment to be correct, and if a rainfall of uniform intensity and unlimited duration falls on a basin, the runoff rate per acre will reach a maximum $q = \frac{Q}{A} = K i$ at time T_c and then remain constant. Thus conceived, K represents the ratio of q to i. If the rainfall should stop at time T, the runoff would fall to zero during time T_c, Fig. 4.

Rainfall seldom occurs uniformly in nature and this difficulty is overcome by stating that it is only the average rainfall intensity during the time of concentration and further correction is added for the variation in the size of the area over which the rainfall is taking place. The following Figures explain the concept of this average i for T_c greater than one hour and T_c

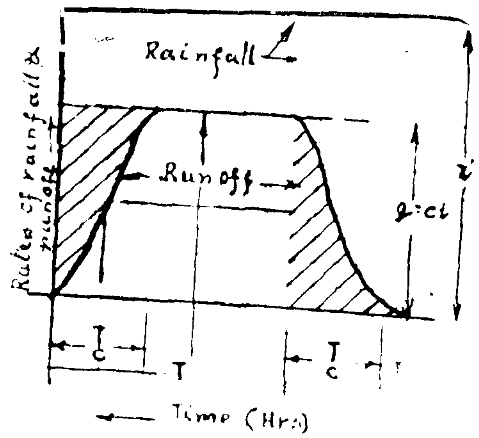


Fig. 4

less than one hour and (i) shown corrected for variation in time and further amended for the variation in the size of the area, Fig. 5 (a) and (b).

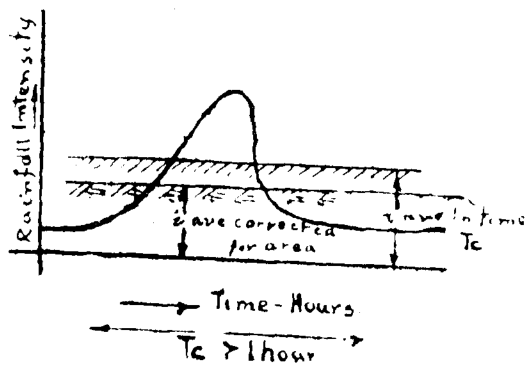
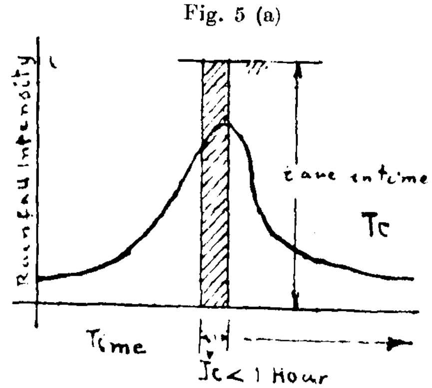


Fig. 5 (a)



The detailed determination of factors like the variation of intensity of rain in time and area, the time of concentration and the coefficient K to forecast the maximum flood that could be expected from a particular catchment under study, are each explained in subsequent sections.

3. RAINFALL AS A FUNCTION OF TIME

3.1. In this section, rainfall from typical cloudburst is analysed for its variation in "time" for catchments in Andhra Pradesh.

3.2 If the rainfall during a storm is continuously recorded at a given point, it will be found that the intensity varies, generally rising to a maximum and falling off again. This could be seen from Fig. 6 about the rainfall recorded at Begumpet aerodrome from 31st July 1954 to 1st August 1954.

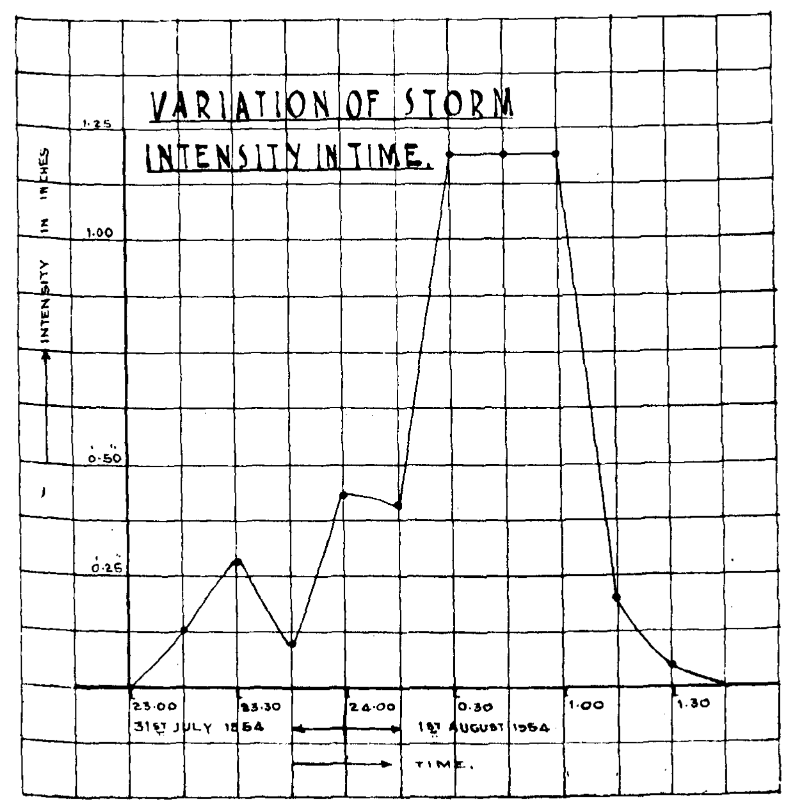


Fig. 6

Readings taken at 15 minute intervals

TABLE 7

Date	Time	Amount of rainfall during the interval (in.)	Percentage
31–7–54	23.00 to 23.15	0.13	2.85
	23.15 to 23.30	0.28	5.60
	23.30 to 23.45	0.09	2.00
	23.45 to 24.00	0.43	8.20
1–8–54	00.00 to 00.15	0.40	7.80
	00.15 to 00.30	1.20	22.90
	00.30 to 00.45	1.20	22.90
	00.45 to 01.00	1.20	22.90
	01.00 to 01.15	0.20	3.90
	01.15 to 01.30	0.04	0.95
		5.17	100

3.3. This may be due to the movement of the storm, the highest intensity being recorded when the centre of the storm is over the measuring point. *In either case, the average intensity of the rainfall during the whole period of the storm may be taken to be some fraction of the maximum intensity.*

3.4. The importance of assessing the rate of rainfall and the frequency of its occurrence has been analysed by many investigators. Each investigator has produced his own formula which is derived from the data of a specific locality. The constants vary from place to place. The usual formula according to Forster [2] is of the form :

$$i = \frac{F}{t} = \frac{R}{t + a} \qquad \text{...... (3)}$$

$$or \quad = \frac{R}{(t + a)^e} \qquad \text{...... 3 (a)}$$

where

i = average intensity of rainfall during the period of the storm at the point of maximum rainfall in inches

F, t = total rainfall of the storm in inches in time t

a = a constant

R = a rainfall coefficient which is constant for any particular storm

3.4.1. From the above equation any one factor may be arrived at if the other two factors are known. Taking the formula in its simple form, i.e., $i = \frac{R}{t + a}$, the intensity (i') of a storm at any time interval (t') could be found out if t', t, i and a are known for that storm by :

$$\frac{i}{i'} = \frac{\dfrac{R}{t + a}}{\dfrac{R}{t' + a}}$$

$$\text{i.e.,} \quad i' = \frac{t + a}{t' + a}\, i \qquad \text{... (4)}$$

3.4.2. Generally, it may be seen from the graphs of many storm studies that if the hourly intensity at any particular place is known then it will be possible to find the intensity of rainfall in any smaller or larger time interval t provided the factor a is available by modifying the equation (4) in the form.

$$i = \frac{1 + a}{t + a}\, i \ \text{(one hour)} \qquad \text{... (5)}$$

In order to get the value of a, data of the same rainfall intensity measured at two different time intervals should be available so that knowing i, i', t and t', the value of a could be estimated. The hourly intensity has been chosen, because the peak intensity in a big storm usually occurs within an hour (Fig. 20) and is sustained in that interval. After this the rainfall generally flattens out. In bigger storms this intensity may last longer as, for example, for one hour and forty five minutes during the rain of October 1958, analysed in para 3.8. In such cases the average intensity will have to be worked out taking that duration into consideration to get the correct discharge. The effect of this longer duration is shown in Fig. 7 and the effect of this is

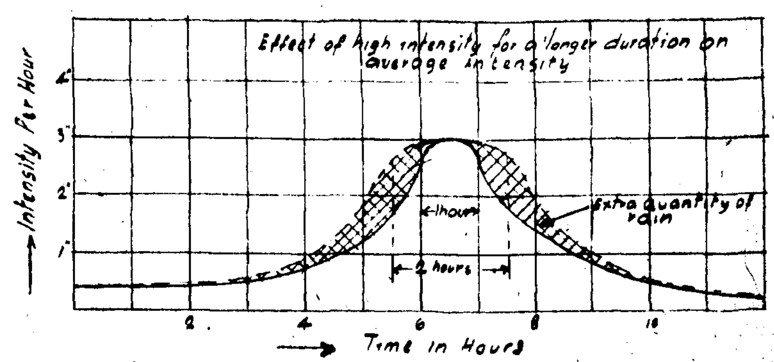


Fig. 7

demonstrated in *Example 3* given at the end. This is observed only in very big storms, the effects of which cause great havoc, and this point should, therefore, be kept in view while working out the highest flood discharge for catchments having a rainfall of 12 in. per day and above.

3.4.3. The intensity of the same storm measured at one hour and 15 minute intervals obtained from automatic rain gauges located at Begumpet and Visakhapatnam aerodromes has been collected and the values of a have been worked out in Table 8 for typical storms observed over Andhra Pradesh. These have been worked out for time intervals of less than one hour and greater than one hour as a is found to be varying differently in shorter intervals of time as compared to a longer duration.

TABLE 8

Date		Maximum hourly intensity (inches)	Maximum hourly intensity based on 15 minute intervals (inches)	Value of a	
				Less than one hour	Greater than one hour
Begumpet					
16th August 1955	...	1.16	1.80	1.1	0.52
12th August 1955	...	1.12	1.84	0.97	2.80
24/25th July 1953	...	1.75	3.24	0.63	0.87
31st July 1954 / 1st August 1954	...	4.00	4.80	3.60	1.23
20th July 1940	...	1.66	4.16	0.25	0.14
26/27th June 1943	...	0.94	2.42	0.225	3.85
24/25th June 1944	...	1.28	1.84	1.47	1.17
25th July 1945	...	0.87	1.92	0.37	2.95
11th November 1948	...	1.78	2.72	1.17	—
10/11th July 1950	...	1.03	1.96	0.58	1.11 or 2.6
12th September 1959	...	0.80	1.44	0.84	—
24th July 1953	...	1.66	3.12	0.30	2.5
Visakhapatnam					
23rd July 1951	...	3.36	9.12	5.35	Not available
20th August 1951	...	2.20	4.00	0.67	do.
22nd September 1951	...	2.11	2.80	2.06	do.
28th September 1950	...	2.03	4.64	0.33	do.

3.4.4. Graphs to show the variation of a with change in intensity of rain have been plotted, Fig. 8. Where the value of a has been plotted as a function of the intensity for catchments with time factor of less than one hour, the value of a does not

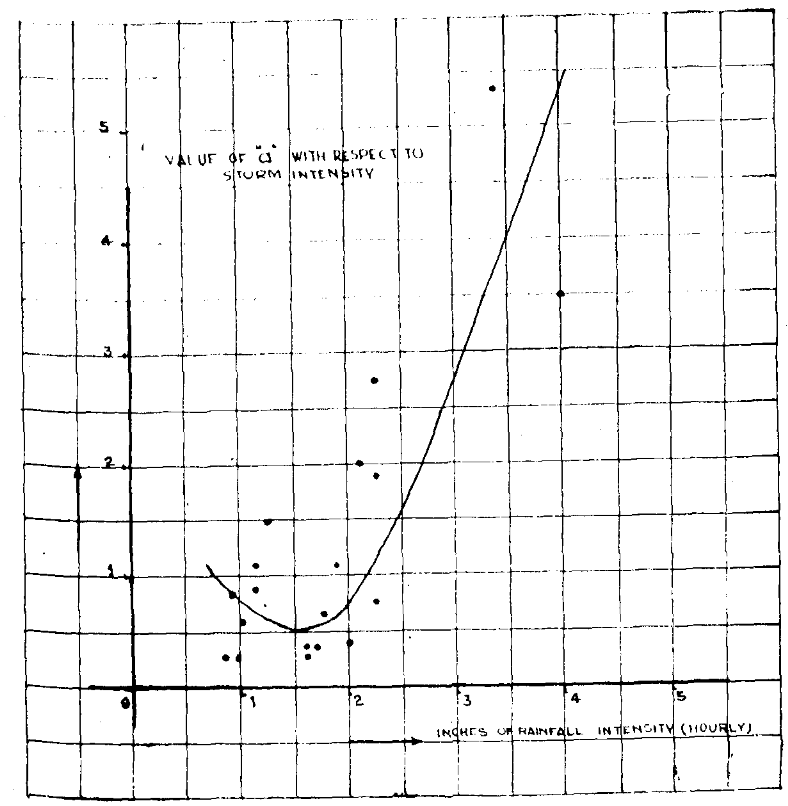


Fig. 8

follow a straight line variation. The value of a for working out the average intensity for such catchments should be taken from this graph. The values of a suitable for various intensities found to cover most storms in Andhra Pradesh where the time factor for catchments is less than one hour are given in Table 9.

3.4.5. From Fig. 9 where the value of a has been plotted against different intensities for storms of greater than one hour duration applicable to catchments having a time factor of more

TABLE 9

Hourly intensity in inches	Value of a proposed for T_0 less than one hour
1.50	0.50
1.75	0.75
2.00	1.00
2.50	1.50
3.00	2.50

than one hour, it may be seen that, except in low ranges, the value of a is practically the same and can be taken to be 'one'.

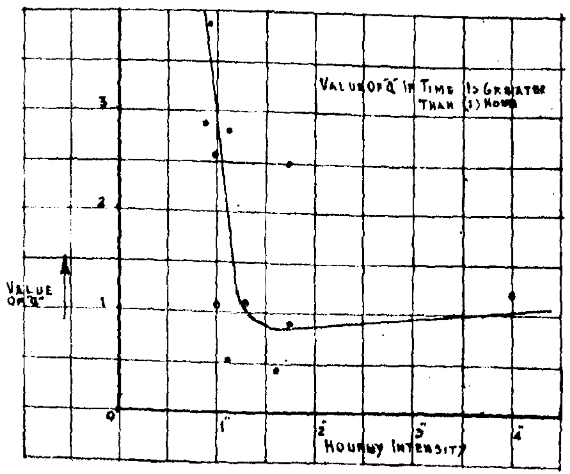


Fig. 9

3.5. Zones of Equal Hourly Intensity

The rainfall data on the basis of hourly intensities is not generally available. As all variations of storms are worked out on the basis of hourly intensity, this could only be assessed from a study of similarity of areas and by comparing an unknown area to the characteristics of a known area. From the known variations of storms at the two automatic rain gauge stations in Visakhapatnam and Begumpet, the rainfall variations at other places have been analysed on the following basis:

(1) The relation between the average intensity to hourly intensity at places where the amount and duration are known.

(2) The relation between the daily rainfall to the hourly intensity at places where only the total daily rainfall is known.

(3) Working back on the known discharges and fixing intensity on this basis.

(4) Contiguity of the areas from a study of isohyetals.

A map of Andhra Pradesh, Plate I, has been prepared from iis analysis dividing it into zones of equal hourly intensity from hich values of a and i can be selected for purpose of getting the ood discharge.

3.6. Ratio of Hourly Intensity to Average Intensity

To study the significance of the average intensity of known torms whose duration, amount and maximum hourly intensity re given in Table 10, a graph has been plotted, Fig. 10. The X xis gives the duration of the storm and the Y axis the ratio of ourly intensity to average intensity. From this it will be seen hat this ratio increases with the duration of the storm. A long lrawn out drizzle need not produce a burst of high intensity of ain.

TABLE 10

S. No.	Date	Duration of storm hours	Total rain during that time inches	Average per hour inches	Maximum hourly intensity recorded inches	Ratio : Maximum hourly / average
1.	16-8-55	3	1.57	0.50	1.16	2.32
2.	12-8-55	7	3.11	0.44	1.12	2.90
3.	24-7-53	5	2.79	0.56	1.75	3.10
4.	31-7-54	9	7.85	0.87	4.00	4.60
5.	20-6-40	4	2.15	0.53	1.66	3.15
6.	28-6-43	3	2.56	0.52	0.94	1.80
7.	24-6-44	5	2.27	0.45	1.28	2.90
8.	25-7-45	3	1.58	0.53	1.03	1.97
9.	11-7-54	5	2.26	0.55	1.03	1.97
10.	12-9-50	7	2.60	0.27	0.86	2.32

3.7. The maximum hourly intensity that could be expected from a rainfall of the order of 12 in. per day or near about was made from a study of the heavy rainfall that occurred during

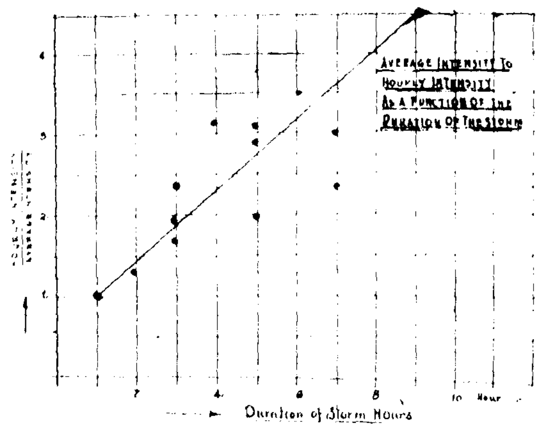


Fig. 10

the 19th and 20th October 1958 near coastal Andhra. An isohyetal map showing the extent of the storm on that day—Plate II (d), has been prepared from a study of the rainfall recorded at different places in the region.

3.8. A graph taken out of the automatic rain gauge showing the rainfall as it occurred during that period is given in Fig. 11. This rain caused floods which were responsible for the collapse of many road bridges, Photographs 1, 2, 4 & 7, breaches in the railway line and inundation of the Visakhapatnam town.

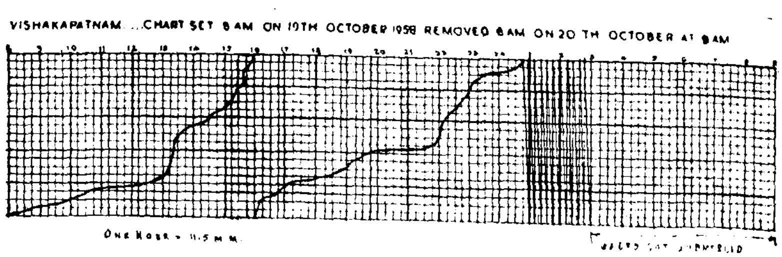


Fig. 11

An analysis of this rainfall reveals that during a period of one hour, the intensities work out as follows:

(i) Maximum one hour intensity

The average intensity of rainfall from 01.00 a.m. to 2.45 a.m.:

1. Rainfall 108.76 mm
2. Time 1 hr. 45 minutes

$$\text{Therefore hourly intensity} = \frac{108.76 \times 4}{7}$$

$$= 2.45 \text{ in. per hour.}$$

(ii) Maximum intensity

This occurred from 02.23 to 02.30 hours on 20.10.58. It lasted for seven minutes. During that time:

1. Rainfall in 7 minutes 10.16 mm
2. Rainfall per hour $= \dfrac{10.16 \times 60}{7 \times 2.4 \text{ in.}}$

$$= 3.43 \text{ in. per hour.}$$

The above results give a value of $a = 2.1$ for 2.5 in. hour intensity.

3.9. The total rainfall in one day taking the figures from the ordinary rain gauge after the automatic rain gauge was put out of order due to submergence by floods may be taken to be 13.3 inches. Thus an intensity of 2.5 in. per hour could be taken as the intensity that might be expected in areas which receive a rainfall of more than 12 in. a day, and could have a duration of nearly one hour and 30 minutes.

3.10. It is observed by examples of flow of discharges worked out by area velocity methods of streams in Adilabad and Mehabubnagar Districts that for equal areas the streams in Adilabad District gave 40 per cent more yield considering that K and A are the same. That will give an intensity of $\dfrac{1.75 \times 140}{100} = 2.5$ in. per hour in Adilabad District. Considering that 12 in. is the rainfall in a day recorded in that zone, this is a good confirmation of the assumption made in para 3.9.

3.11. The other boundaries of the zones have been drawn keeping in view the isohyetal lines, total rainfall and average intensities, and these boundaries corrected after working out actual examples in the zones. The use of this map has been illustrated by working out examples based on this data, and suitable values for designing cross drainage works in the zones could be selected from this.

3.12. The rainfall collected and plotted at each place is the maximum recorded in any day as far as data is available. This has been taken into account considering that it indicates the possible high contour of an isohyetal that could position itself on that area and produce a heavy shower. The definition of isohyetals and other area characteristics of a storm are explained further.

4. RAINFALL AS A FUNCTION OF AREA

4.1. According to Richards[1], if the rainfall during a storm is measured at a number of points affected by it and points of equal rainfall (isohyetal lines) are plotted, the result will be a chart resembling a contour map. Isohyetal lines for average intensity of rainfall on a rainy day for three typical areas of Andhra Pradesh, viz., (1) Telengana, (2) Rayalaseema and (3) Coastal Andhra, have been prepared (Plate II, a, b, c, and d). There will be small areas of maximum rainfall with surrounding areas of gradually decreasing rainfall defined by the isohyetal lines and the average rainfall over the whole area will be some fraction of the maximum spot intensity at the focus of the storm.

4.2. The maximum intensity of rainfall as could be seen from an isohyetal map occurs at a point over a small area at the centre of the storm. Outside this, it gradually decreases so that the average intensity over a larger area becomes less with increase of this area. Various factors such as air currents and irregularities of the ground lead to uneven distribution of the rainfall as could be seen from the isohyetal maps given. In large storms of long duration the isohyetal chart may show more than one summit under such irregular conditions. It can be inferred from these charts that the ratio of the average rainfall over an area to the maximum rainfall decreases with increase of the area. To plot the isohyetal lines would require the collection and analysis of synchronised readings of rain gauges suitably located over the storm area in sufficient numbers to ensure that a suitable isohyetal chart of a representative storm is obtained. From this chart the average rainfall over the area up to each isohyetal can be computed. Plotting these averages against the corresponding areas, diagrams will be obtained which will show the average intensity of rainfall over areas upto the full area of the storm. Synchronised rain data of fairly heavy known storms in Telengana, Rayalaseema and Coastal Andhra collected and

plotted in Plate II (a, b, c, d) have been used to get curves showing the spread of the storm as a function of the area, Fig. 12. These curves could also be prepared for each individual district

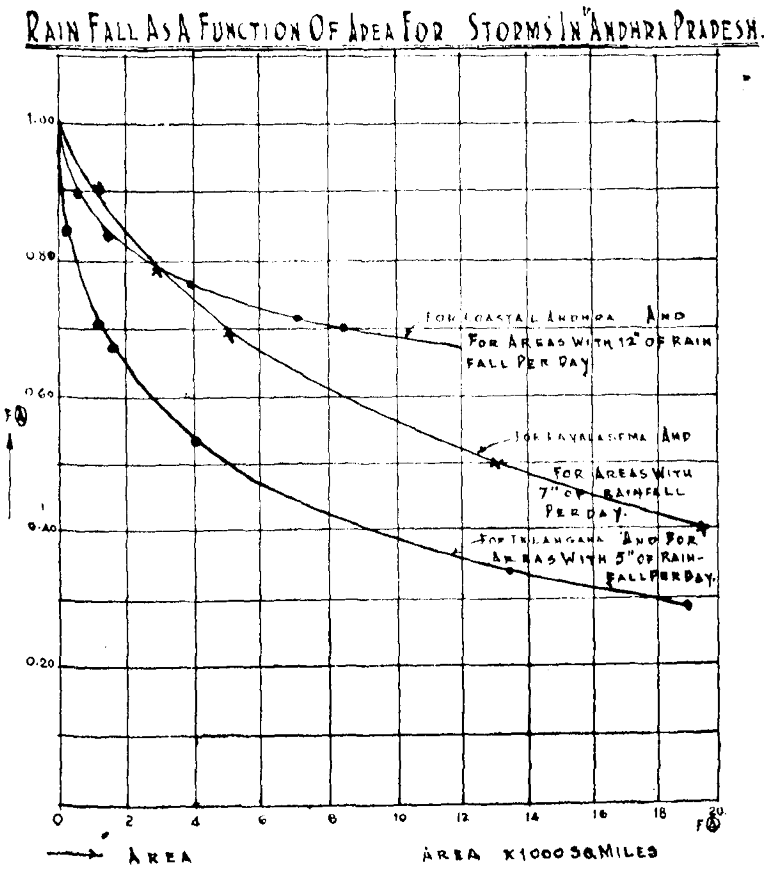


Fig. 12

if sufficient number of rain gauge stations were available in the districts. Unfortunately, there are no more of these rain gauges than the number of talukas in each district and, therefore, use has to be made of the data collected and interpreted on zonal basis.

4.3. The areas of storms and the ratio of maximum intensity to average intensity have been computed and given in Tables 11 to 13.

TABLE 11

Telengana

Storm occurrence, 26th June 1956

Intensity of rainfall (inches)	Area of storm in sq. miles	Progressive total area of storm in sq. miles	Total rainfall i. A. (in. miles)	Progressive total of Column C	Average intensity	Ratio with maximum intensity
i	A	B	C	D	D/B	f (A)
5.50	40	40	220	220	5.50	1.000
4.50	288	328	1300	1520	4.70	0.850
3.50	1128	1456	3950	5470	3.75	0.690
2.50	2552	4008	6420	11890	3.00	0.550
1.50	9564	13572	14400	26290	3.94	0.355
0.88	5372	18944	4700	30909	1.62	0.295
0.62	7276	26220	4413	35403	1.36	0.250
0.37	14816	41036	5500	40903	1.30	0.235
0.15	10400	51436	1300	42203	1.02	0.185

TABLE 12

Coastal Andhra

Date of Storm - 18th November 1923

Intensity of rainfall (inches)	Area of storm in sq. miles	Progressive total area of storm in sq. miles	Total rainfall i. A. (in. miles)	Progressive total of column C	Average intensity	Ratio maximum intensity/average intensity
i	A	B	C	D	D/B	f (A)
15.00	12	12	186	186	15.5	1.00
14.50	96	108	1392	1578	14.6	0.94
13.50	220	328	2970	4548	13.9	0.89
12.50	1140	1468	14250	18798	12.8	0.83
11.50	2600	4068	29900	48698	12.0	0.77
10.50	2000	6068	21000	69698	11.5	0.74
9.5	1264	7332	12508	82256	11.2	0.72
8.5	1300	8632	11050	93256	10.8	0.70

TABLE 13

Rayalaseema

Date of storm - 21st May 1940

Intensity of rainfall (inches)	Area of storm in sq. miles	Progressive total area of storm in sq. miles	Total rainfall i. A. (in. miles)	Progressive total of column C	Average intensity	Ratio maximum intensity/average intensity
i	A	B	C	D	D/B	f (A)
6.50	40	40	260	260	6.50	1.00
5.50	1048	1088	6000	6260	5.98	0.92
4.50	1376	2464	6200	12460	5.10	0.79
3.50	3232	5696	11300	23760	4.86	0.75
2.50	7164	12860	17800	41560	3.24	0.50
1.50	9620	22480	14500	56060	2.50	0.30

4.4. These figures when plotted will give rainfall as a function of area as in Fig. 12.

Fig. 12 suggests that the distribution of cloudbursts and rainfall is more extensive and uniform in coastal areas of Andhra Pradesh than either in Central Telengana or in Rayalaseema.

Thus the intensity 'i' in equation (1) after the discussions in sections 3 and 4 gets modified as a function of time and area as follows:

$$\text{av. } I = f(t, A)\, I \text{ maximum.}$$

5. TIME FACTOR FOR A CATCHMENT AND ITS RELATION TO DISCHARGE

5.1. It is clear from equation (3), given in para 3.4, that time has effect on the average intensity of rainfall on a catchment. Each catchment depending upon its size, shape and slope has a time factor in which the full effect of rainfall over it would be felt. Determination of this time factor is necessary to modify the known hourly intensity over that area to suit the effective time for the whole catchment for calculating the runoff.

5.2. This time factor has been described by B.D. Richards[1] as "Time of Concentration." This is the maximum time required for the rainwater from all the points on the boundary of the catchment to reach the point of concentration—either a bridge or a culvert. This time depends upon :

(i) the length of travel of the rain from the boundaries of the catchment ;

(ii) the slope of the catchment ;

(iii) the type of soil in the catchment ; and

(iv) the intensity of rainfall that could be expected on the catchment.

5.3. Considering a catchment of length L, slope S and of unit width, exposed to a uniform and continuous rainfall of average intensity i, Richards obtained an equation for the time of concentration.

5.4. He assumed the slope from its head at a point 'A' to the point of concentration 'B' to be 'S'. With the advent of storm, rain would commence to fall over the catchment. The water reaching 'B' will at first be only that falling in its vicinity, but it will gradually increase as water from all other points on the catchment arrives until, finally, when the water from 'A' arrives in time T_c after the commencement of storm, the point 'B' will be receiving the flood contributed by the whole catchment.

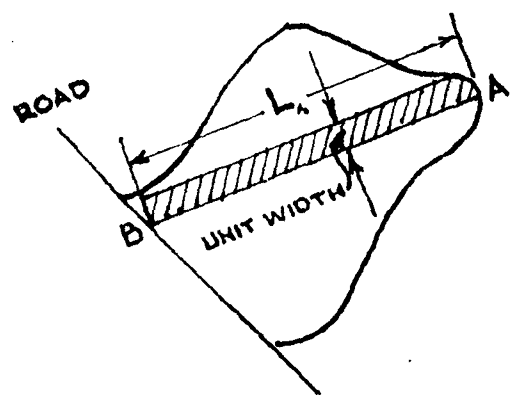


Fig. 13

5.5. Assuming the depth of flow at $A = O$ and hydraulic mean radius $= r = i$ (as the depth of flow is small), the average velocity will be equal to 2/3rd the maximum. From this,

Photo 1.
Breaches to the road left, railway tracks and fencing by floods in Visakhapatnam

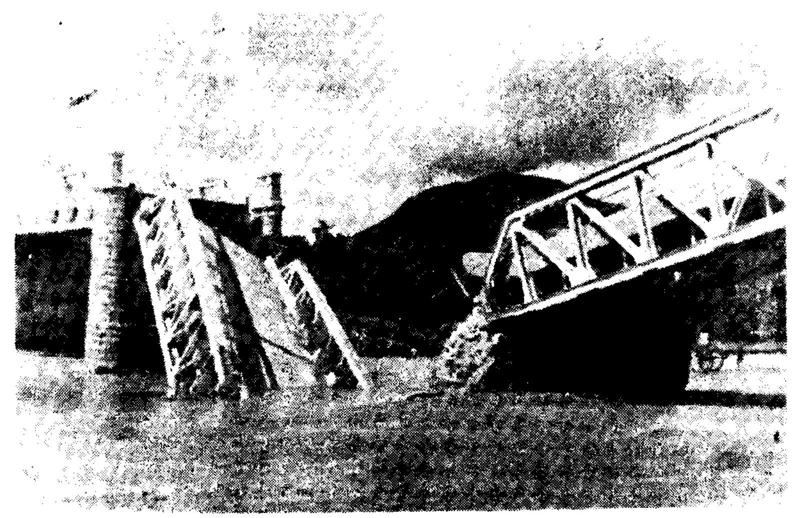


Photo 2.
Kandivalasa Bridge on N. H. 5. Pier tilted due to scour during a heavy flood and two spans collapsed

Photo 3.
Breach of dam by overtopping under extraordinary floods

Photo 4.
Overturning of a bridge in extraordinary floods

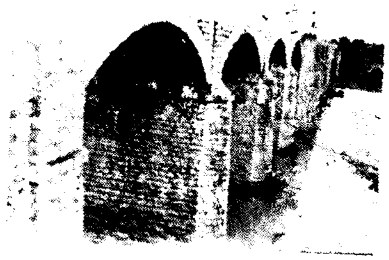


Photo 5.
The spandrel filling washed away by overtopping. This also shows the strength of an arch bridge to withstand such onslaughts

Photo 6.
The distance of the masonry blocks of Kadam dam upstream of the above bridge carried down during the same flood

Photo 7.
Collapsed bridge at M. 474/8 on N. H. 5 under repair.
Note the fallen down bus in the back ground

Richards[1] derives a formula for the time of concentration T_c as:

$$T_c^3 = \frac{9}{4\,C^2} \cdot \frac{L^2}{K.i.S.} \qquad \dots\dots\dots (6)$$

where K = coefficient of runoff

C = coefficient given by Bazin.

5.6. Dawson[5] in his Paper "Discharge from Heavy Rainfall" arrives from similar assumptions: but considering residue on the catchment: upstream of the point B:

$$t = \frac{L^{\frac{2}{3}}}{C_1^{\frac{2}{3}}\, i^{\frac{1}{3}}}$$

where $C_1 = C\sqrt{S}$

$$\text{If slope} = \frac{\text{fall of catchment in feet}}{\text{length of catchment in miles}} = \frac{H}{L}$$

then both will reduce to:

$$T^3{}_c = \frac{9}{4} \cdot \frac{5280}{C^2 K.i} \cdot \frac{L^3}{H} \qquad \dots (7) \text{ Richards}$$

$$t = \frac{5280}{C_1{}^2 i} \cdot \frac{L^3}{H} \qquad \dots (8) \text{ Dawson}$$

where H is in feet.

Both could be written in the form:

$$T_c = \left(\phi\, \frac{L^3}{H}\right)^{\frac{1}{3}} \qquad \dots (9)$$

$$t = \left(\theta\, \frac{L^3}{H}\right)^{\frac{1}{3}} \qquad \dots (10)$$

According to Dawson, it is not necessary to take the time of concentration to be that time which is taken by the water to flow from the farthest point to the point of concentration as is implicit in Richards' assumption, but he proves that maximum flood will have occurred by the time the water has reached 0.544th of the distance of the farthest point of the catchment.

5.7. The assumption made in both the methods is that the catchment is sloping uniformly in one direction for an infinite width. This does not take into account that, in practice, the catchments will not give wholly unidirectional flow, but will be in a series of valleys with cross, angular and meandering flows. For all these factors to even out, which is very complicated to determine mathematically, the time of concentration as given by Richards has been taken for the purpose of this Paper as the time by which the flood could be considered effective. Considering that the actual length of flow taken along the meandering length as compared to straight length may be nearly double, this assumption is not far out from reality.

5.8. The value of ϕ is not a constant but as could be seen from equations (7) and (8), it depends on :

(1) the intensity of rainfall on the catchment,

(2) the nature of the soil forming the catchment.

5.9. The effect of this combination is not easy to evaluate except by experiments. This value can be obtained by measuring the time taken by a float or some such device to travel a known distance and slope for a particular intensity of rainfall for a particular type of soil. If this is obtained, then the value of ϕ could be obtained by substituting in the equation :

$$\phi = \left(\frac{T_c}{L}\right)^3 H \qquad \text{... (11)}$$

5.10. Experiments to ascertain the value of ϕ for different types of soils and intensities of rainfall were formulated. Suitable catchments for observation were selected and their length and slope were determined and the soil type noted. During a downpour the intensity of which was measured by the amount of water collected on an open pan, floats or oil were let down at the source and the time taken by them to traverse the distance was noted. This gave directly the time of concentration for a particular intensity of rainfall. Where such opportunity was not available, the value of 'C' was determined and from this ϕ was computed by approximation. The value of n to determine C cannot be taken from values given by Manning as depths of flow are small and the boundaries vast compared to a stream or canal. As is well known, the value of n is not a constant but is a function of depth of flow, shape of flow, silt charge, Reynold's number and other factors.

Experiment 1

Location : Stream in mile 17/7 of Lingampalli-Hyderabad road (28th January 1958).

Type of catchment : Rocky gravel mixed with moorum and sand,

Intensity of rain : 2 in. per hour

Average H. M. D. of stream = 0.6 by measurement

Length L = 11,728 ft

Fall H = 38.72 ft

Slope $S = \dfrac{38.72}{11728} = 0.0033$

To find the value of C in $V = C\sqrt{rs}$

Time taken by the water to flow this distance = 1 hr. 40 min.

$$\text{Velocity} = \frac{11728}{100 \times 60} = C\sqrt{0.6 \times .0033}$$

$$\therefore \quad C = 14$$

$$\phi = \left(\frac{1.66 \times 5280}{11728}\right)^3 \times 39$$

$$= 16 \text{ for 2 in. intensity per hour.}$$

Experiment 2

Location : Stream near Begumpet aerodrome

Type of soil : Sandy clay

Average H.M.D. of the stream = 2.08

Length L = 810 ft

Fall H = 1.6 ft

Time taken by the float for the water to flow this distance = 12.5 minutes

C = 17.

Experiment 3

Type of soil : Black cotton mixed mostly with rocky soil

Average H.M.D. = 2.4

Length L = 5 miles = 26,400 ft

Fall H = 300 ft

Time taken by the float = 3 hours 20 minutes

C = 13

(Estimated intensity of rainfall 2 in. per hour).

Results of experiments on stream in mile 161/5 of National Highway in Adilabad District carried out on two separate occasions were also obtained. On the first occasion by the time the origin of the stream was reached, the rain stopped at the source and the flood started receding. At the second attempt the rainfall was full and heavy with an estimated intensity of 2.5 in. per hour. The soil of the catchment is heavy black cotton.

TABLE 14 (a)

	L in miles	H in ft	T_c in hours & minutes	ϕ	Estimated intensity in. per hour
Stream in mile 161/5 of National Highway No. 7 (North)					
1st experiment	5.00	388	3—33	8.7	1.5
2nd experiment	5.52	333	1—40	3	2.5

From these experiments and based on the results of problems worked with these coefficients, the values of ϕ for purposes of arriving at the time of concentration with different intensities of rain have been fixed and given in Table 15. With more results becoming available, this Table would be made more broad based to include other kinds of soil like laterite, etc. Results obtained at a few more places are given in Table 14 (b).

TABLE 14 (b)

	L	H in ft	T_c in hours & minutes	ϕ	Estimated intensity and soil
1. Nala at mile 31/4 on Yerigiru Ijj Alampur road	5800 ft	47	0—14	0.73	Black cotton soil 3 in. per hour
2. Causeway in mile 74/4 of Hyderabad-Hanumakonda road	4.5 miles	97	1—50	6.2	Black cotton soil mixed with red earth; 2 in. per hour
3. Stream in mile 110/2 of Hyderabad — Chanda road	1.25 miles	43	0—19	0.7	Soudu; 3 in. per hour.
4. Chelgal stream on Jagtial-Armur road	3.5 miles	223	0—35	12	Moorum; 2 in. per hour.

These values have given satisfactory results when applied to similar catchments.

TABLE 15

Value of ϕ in equation (11)

Intensity of rain per hour in inches	Hard soi with rocky outcrop	Soft soil like sandy gravel with grass and close vegetation	Black cotton soil
1.75	19.00	28.00	9.00
2.00	14.00	21.00	6.00
2.50	11.00	12.00	3.00
3.00	8.00	10.00	2.00

5.11. The values given in Table 15 can be combined to suit different soil distribution to arrive at the time of concentration. The catchments are generally not in *one* uniform slope. They are mostly in two or three distinct slopes like the steep slope in mountains or hills with a flatter slope in the plains.

5.12. Catchments with Varying Slopes

While calculating the time factor, it has been taken that the velocity at the beginnig is zero, i.e., the H.M.D. is nil at the start and increases as more water gathers. In order to keep the same assumption while calculating the time of concentration for catchments with varying slopes, the velocity calculations for the different slopes making up the catchment are made from the same origin.

Consider a catchment ABC with a steep slope between A and B and a flatter slope between B and C, Fig. 14. Extend the slope of B and C to X so that XC will have the same slope as B and C, say, S_2.

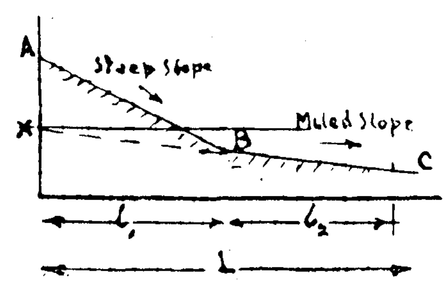


Fig. 14

Let the horizontal distances $AB = l_1$ and $BC = l_2$, then $AC = L = l_1 + l_2$.

The waters from A to B will flow with a velocity due to average H.M.D. between the two points and also due to the slope between them.

Now CB has been produced to 'X' with the same slope as BC. The slope AB being steeper than XB, the velocity at B will be greater if the water travelled from A to B than if it travelled from X to B, i.e., the water from A will arrive at B earlier than from X.

From the point B the water will flow to C with a velocity depending on the slope and H.M.D. between B and C.

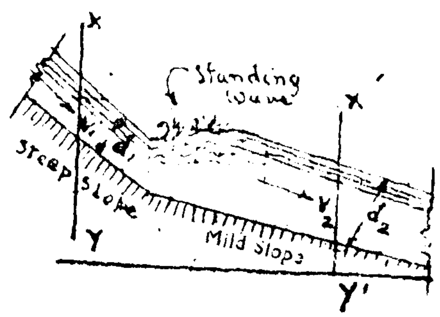


Fig. 15

The excess velocity between A and B is dissipated by a standing wave, Fig. 15, in a short length if the slope is not critical and water flows between B and C with slower velocity but with greater depth. This explains the submergence of many railway and road tracks laid on plains below hilly regions.

5.13. Other things being equal, the velocity is a direct function of the slope. Let BD, Fig. 16, represent the maximum velocity (V max.) due to the slope S_1 at B, let BE represent V' max. at B due to the slope S_2 between A and B and let CG represent V max. due to slope S_2 between A and C.

The difference in velocity due to slopes S_1 and S_2 between A and B will be:

$$BD - BE = DE$$

The time gained by the water in the reach AB due to the steeper slope is therefore $= \frac{l_1}{DE}$.

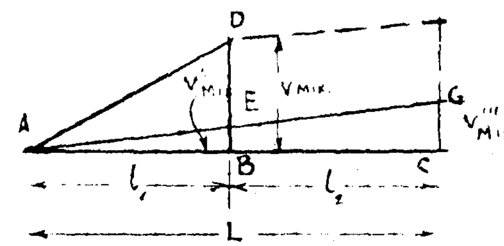


Fig. 16

The time that would have taken for the water to reach the point C if the catchment had a uniform slope S_2 is :

$$= \frac{l_1 + l_2}{GC}$$

$$= \frac{L}{GC}$$

The time of concentration between A and C

$$T_c = \frac{3}{2}\left(\frac{L}{GC} - \frac{l_1}{DE}\right)$$

$$\text{as } V \text{ av} = \frac{2}{3} V \text{ max.}$$

This is due to a net average velocity of

$$\frac{2}{3}(GC + DB - EB)$$

$$\text{or } T_c = \frac{3}{2}\left[\frac{L}{GC} - \left(\frac{l_1}{BE} - \frac{l_1}{DB}\right)\right]$$

$$= \frac{3}{2}\left[\frac{L}{GC} - l_1\left(\frac{1}{BE} - \frac{1}{DB}\right)\right]$$

This procedure can be extended to any number of significant slopes in the catchment.

The procedure of getting the time of concentration will, therefore, be:

(1) Work out the time of concentration between A and C due to flat slope between B and C extended up to X.

(2) Deduct from the above the difference in time of concentration between A and B and X and B.

The use of this procedure is demonstrated in Example 1 at the end and in the same problem the probable error that may creep in if this factor is ignored is also shown.

5.14. This method of determining the time of concentration takes the shape factor also into account. Fig. 17 illustrates the effect of shape on the length of stream and consequently on the time of concentration which, in turn, is used to determine the average intensity over the whole catchment.

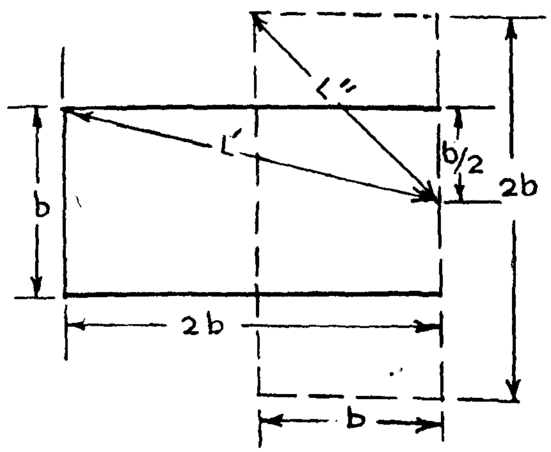


Fig. 17

Here L' and L'' vary according to a and b which determine the shape of the catchment.

5.15. Richards takes the time of concentraion as the time taken by the water from the farthest point on the catchment to reach the point of concentration. This may be true in case of a uniformly sloping catchment but in hilly catchments mixed with plains, the water from the farthest point with a steep fall may reach the point of concentration very much earlier than a point which is nearer but with a flat slope. Therefore, it is necessary in such catchments to try and obtain a few values of 'T_c' to decide the effective time of concentration for that catchment.

5.16. From the discussion so far the runoff from a catchment can, therefore, be expressed in the form:

$$Q = K.A.i.$$
$$= K.A.f\,(A,T_c,i)$$
$$= K.A.\left[f'(A) \frac{1+a}{\left(\phi \frac{L^3}{H}\right)^{1/3} + a} .i \right] \quad \ldots \quad (12)$$

5.17. An inconsistency to be guarded against in this method is in selecting the outer point on the boundary of the catchment. In a catchment of the shape shown in Fig. 18, if the area and the time of concentration for the whole catchment from EF to AB

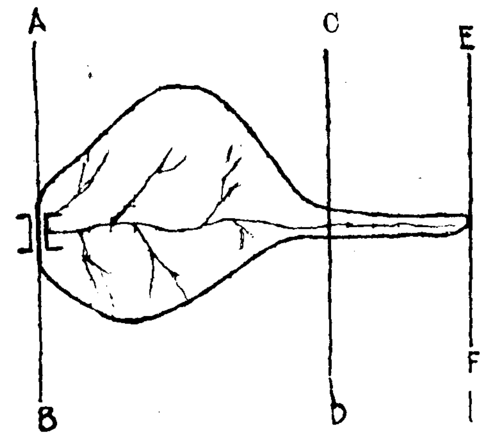


Fig. 18

are A_1 and T_1 respectively and for the cutfall from CD to AB, A_2 and T_2 respectively, then the discharge for the whole catchment between EF to AB is:

$$Q_1 = K.\ A_1\ i_1$$

Here i_1 depends upon T_1 which being longer will give less mean intensity over the whole area, whereas in $Q_2 = K\ A_2\ i_2$, T_2 depends on outfall from CD to AB and will give a higher intensity. The ratio of intensities i_1/i_2 may not be in the same ratio as A_1/A_2 in all such cases. It is, therefore, suggested that such narrow strips of catchments may be particularly taken care of.

6. DETERMINATION OF THE COEFFICIENT OF DISCHARGE 'K'

6.1. The determination of the coefficient of discharge is a very important factor as any error in this regard will directly affect the quantity of runoff. In bridges and culverts where the maximum runoff that could be expected from a catchment is required, the value of K should be kept on the high side unlike the design of storage works where the average value or moderated value or the integration of the unit hydrograph is desired.

6.2. The runoff is the nett rain that comes out of a catchment after allowing for losses like absorption and evaporation from the catchment.

6.3. The effect of the nature of the soil on the yield is taken into account while determining the time of concentration for the catchment. The nature of the ground influences the time of concentration which, in turn, influences the effective 'i' taken for the calculation of the runoff.

6.4. The amount of losses that can be expected by absorption and evaporation has been worked out by different workers. Two examples, (1) by Strange and (2) work done on small plots in America given by Forster are given below. From these it may be seen (Table 16) that the percentage yield increases as the quantity of rain increases.

TABLE 16

Daily rainfall in inches	Original state of ground		
	Dry yield per cent	Damp yield per cent	Wet yield per cent
0.25	—	—	8
0.50	—	6	12
0.75	—	8	16
1.00	3	11	18
1.25	5	14	22
1.50	6	16	25
1.75	8	19	30
2.00	10	22	34
2.50	15	29	43
3.00	20	37	55
4.00	30	50	70

A graph plotted from the above values is given in Fig. 19. About 70 per cent is the maximum yield that can be expected from a continuous rain of 4 inches.

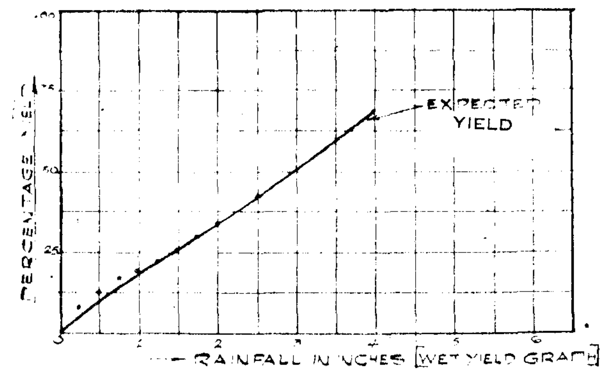


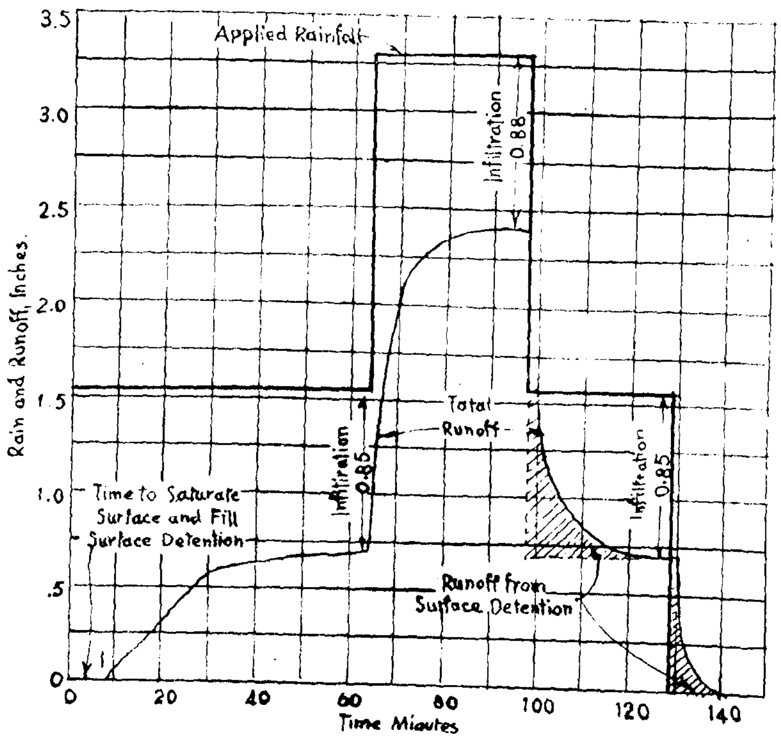
Fig. 19

6.5. The effect of quantity of rain on yield from catchments by simulated rainfall induced on small experimental plots is given by Forster. The simulated rainfall was moderately heavy, i.e., 1.55 in. per hour. Some seven minutes were required to saturate the surface and overcome the surface detention before the runoff started. The runoff gradually increased for 50 minutes by which time a steady rate of 0.70 in. per hour was reached. This rate continued until the rate of rainfall was increased to 3.30 in. per hour whereupon the runoff rose to 2.42 in. per hour giving a yield of 73 per cent.

6.6. Thus it appears that the loss is practically the same (0.85 in. to 0.88 in.) for any quantity of rain falling and the percentage yield increases with the quantity of rain and could be selected from the graph given in Fig. 19 according to the amount of rainfall on that area at any one time. In fact, it has been observed by the workers in this field that the loss due to absorption per hour goes on decreasing from 0.8 to 0.7 in. per hour and then to 0.6 in. per hour and so on, giving more and more yield as the duration of rainfall continues to increase.

6.7. In addition, it may be expected that if the intensity of rainfall is great the water will flow at a faster rate and some of it may not get absorbed or evaporated or held to the extent it would have done if it had flowed at a lesser speed. To what extent the extra yield will be on account of these factors is a matter for experimentation.

6.8. To find out the effect of rainfall intensity on yield under field conditions, a small catchment was selected as near as possible to the automatic rain gauge recorder at Begumpet to get the intensity of rain falling on the catchment. While selecting



Rainfall and Runoff from a Plat Experiment From Sharp and Holtan

Fig. 20

the location of the catchment it was chosen to be in the direction of the storm passing over the automatic gauge to ensure that the recording was true for the catchment under study.

6.9. The catchment feeding the gauging point was surveyed to determine its area. The area thus calculated was 6.64 acres. The runoff from this area passed through a rectangular culvert of size 3 ft 3 in. wide, 2 ft 6 in. high and 28 ft long.

6.10. The object of the experiment was to compare the actual quantity of rainfall received on the catchment by multiplying the intensity of rainfall (from the readings of the automatic rain gauge) at any given time with the area of the catchment, and the actual yield at the culvert point by gauging the flow. This was done by measuring the width and depth of flow in the culvert and the velocity was determined by recording the time taken by a float (in this case, a ping pong ball) to traverse the 28 ft length of the culvert.

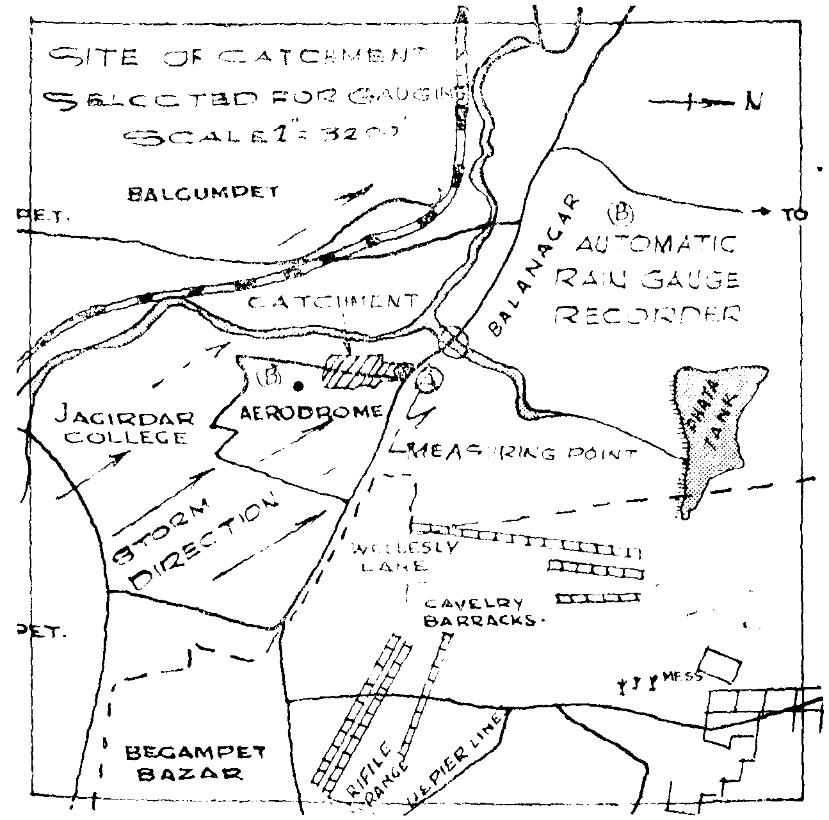


Fig. 21

6.11. The gauging was done on two days—17th and 24th August 1958. The rain was steady for a longer period on 16/17th August, whereas on 24th August it was of short duration but of greater intensity. On both these days the ground conditions were wet.

6.12. On the 16th and 17th August, the rain came in a drizzle of 0.375 in. per hour intensity, the total rain up to the time of measurement being 23 mm or, say 1 inch. This gave a yield of 19 per cent as worked out below:

1. Applied quantity of rain $= 6.64 \times 0.375$
$= 2.49$ cusecs

2. Runoff measured at the gauging point :

(i) Depth of flow $1\frac{1}{2}$ in. $= 1/8$ ft (A steady depth of this height was observed during the whole duration of rain).

(ii) Width of flow = 3 ft 3 in.

(iii) Velocity $\frac{28}{24}$ ft per second

$$\text{Yield} = \frac{1}{8} \times \frac{13}{4} \times \frac{28}{24}$$

$$= 0.47 \text{ cusecs}$$

$$\text{The percentage yield} = \frac{0.47 \times 100}{2.49} = 19 \text{ per cent}$$

This compares well with the expected yield from 1 in. rain given by Strange, Table 16.

6.13. The duration of rain on 24-8-58 was only 10 minutes. From gauge data for the first seven minutes, the intensity was 1 in. and during the next 3 minutes the intensity was 2 in. per hour. The quantity of rain in that period was 0.25 inch.

The percentage yield for these two periods is worked out below:

TABLE 17

Time	Intensity (inches)	Applied rain (cusecs)	Corresponding yield (cusecs)	Percentage yield
13.53 to 14.00	1	6.64	0.95	15
16.00 to 16.03	2	13.28	3.66	28

6.14. The results show the effect of intensity on the coefficient of discharge. For a 0.25 in. rainfall at moderate intensity like $\frac{1}{2}$ in. per hour, the yield that could be expected is about 8 per cent to 10 per cent, Table 16, but at 1 in. per hour the yield is 15 per cent and at 2 in. per hour the yield is 28 per cent. Thus the extra yield due to higher intensity is increasing in an arithmetical progression. From this it would appear that the factor K is a function of both the quantity of rainfall, its time or duration and its intensity and could be expressed in the form :

$$K = f\,(Q.\ i.\ T_c)$$

The measured yield at site during different intervals of time are given below :

Time : hours & minutes	Velocity $= \frac{28}{t}$ ft per second	Depth in inches	Discharge $\frac{3\text{ ft } 3\text{ in.} \times C \times B}{12}$ cusecs
A	B	C	D
16—05	2.8	1.25	0.95
16—08	3.5	2.4	2.28
16—10	3.5	2.4	2.28
16—14	4	2.6	2.8
16—16	4	3.4	$\boxed{3.66}$
16—19	4	2.6	2.8
16—24	4	2.4	2.6
16—32	4	2.1	2.26
16—40	4	1.5	1.62
16—48	3.1	1.2	1.01
17—00	2.3	1.0	0.62
17—12	1.5	0.80	0.32
17—20	0.9	0.70	0.17
17—35	0.8	0.45	0.10
17—45	0.5	0.2	0.027

If the value of K is taken as the yield from Strange's Table 16 for the normal rainfall intensity of $\frac{1}{2}$ in. per hour, it should be increased by an extra percentage to take into account the enhanced yield due to increase in the intensity of rain on the catchment. The extent of extras that are proposed to be added are given in Table 19.

6.15. If the rainfall is 2 in. during one session, then the expected yield for this rainfall from Table 16 is 34 per cent. If this rain has an intensity of 2 in. per hour, then the extra yield that could be expected will be 15 to 20 per cent. Therefore, the final K will be between $0.34 + 0.15$ to $0.20 = 0.49$ to 0.54 which

TABLE 19

	at 1.75 in. per hour	at 2 in. per hour	at 2.5 in. per hour	at 3 in. per hour
Add: To value of K taken from Table 16 according to the amount of rainfall for that place	* 0.05 to 0.15	* 0.15 to 0.20	* 0.20 to 0.25	* 0.25 to 0.28

could be selected from the knowledge of other factors like nearness to the higher or lower intensity zone, etc.

7. CONCLUDING REMARKS

7.1. The rainfall characteristics obtained from different regions of Andhra Pradesh have been collected and analysed to arrive at a reliable flood discharge for designing a bridge or a culvert. These might have to be modified slightly with more data available, but for practical purposes they could be considered reasonably accurate.

7.2. The engineer is interested not only in knowing the *maximum intensity* but also *how often it occurs*. The study of rainfall frequency or return periods is essential for economic reasons. It may not be justifiable to design a drainage work for an intensity of rainfall that may occur, say once in fifty or hundred years.

7.3. The greater the intensity and duration of rainfall, the less frequently it will occur as could be seen from Tables 20 and 21. The intensities adopted for the zones are for economic return periods of less than 12 years as could be made out from Tables 20 and 21. There is no data to compute the return periods of the intensities in all the zones. An idea of these return periods may be formed from a study of the data gathered for the Hyderabad area where such a record is available for 18 years.

7.4. In Table 20, the return periods and their number of occurrence for different intensities are given. They are for a

* Higher values apply to areas nearer to zones of higher intensity.

period lasting half an hour and longer. From this it appears that for this duration, the intensities of $1\frac{1}{2}$ in. to 2 in. per hour occur every year many times. Two in. to $2\frac{1}{2}$ in. per hour occur fairly steadily but, omitting stray cases, could be considered to have a periodicity of five years with two or three years of steady occurrence at a time.

TABLE 20

Begumpet Airport: Number of occasions of different hourly rainfall intensities

Year	$1\frac{1}{2}$ to 2 in.	2 to $2\frac{1}{2}$ in.	$2\frac{1}{2}$ to 3 in.	3 to $3\frac{1}{2}$ in.	$3\frac{1}{2}$ to 4 in.	4 to $4\frac{1}{2}$ in.
1940	1	—	1	—	—	—
1941	1	—	1	1	—	—
1942	1	2	1	—	—	—
1943	4	1	—	—	—	—
1944	3	—	1	1	—	1
1945	3	—	1	—	—	—
1946	1	—	—	—	—	—
1947	1	2	1	1	—	1
1948	1	4	—	—	—	—
1949	1	1	—	—	—	—
1950	—	—	—	—	—	—
1951	4	—	—	—	—	—
1952	—	—	1	—	—	—
1953	2	—	—	—	—	—
1954	—	—	—	1	—	—
1955	—	—	—	—	—	—
1956	—	1	—	—	—	—
1957	1	—	—	—	—	—

Note: These readings are for duration of half an hour and more.

7.5. If a study is made of the return periods of storms with 45 minute duration or above, which is nearer the basis of our flood calculations, the picture alters considerably. It may be seen from Table 21 given for the same years that 1.75 in. intensity may be taken to have a periodicity of 10 to 12 years. Other higher intensities are freak and more data is needed for a longer period to decide about their return periods. By considering that the other zones are contiguous their intensities marked, they could be taken to have a return period of nearly 12 years. This period is considered sufficiently economical for the design of most of our bridges and culverts.

TABLE 21

Returns of storms lasting between 45 minutes to one hour.

Year	0-1 in.	1 to 1½ in.	1½ to 2 in.	2 to 2½ in.	2½ to 3 in.	3 to 3½ in.	3½ to 4 in.
1939	1	1	—	—	—	—	—
1940	—	(1.08″)	—	—	—	—	—
1940	—	—	—	1	—	—	—
1941	—	—	1	—	—	—	—
1942	—	—	—	—	—	—	—
1943	—	—	—	1	—	—	—
1944	—	—	—	—	—	—	—
1945	—	1	—	—	—	—	—
1946	—	—	—	—	—	—	—
1947	—	—	—	—	—	—	—
1948	—	1	—	—	—	—	—
1949	—	—	—	—	—	—	—
1950	—	—	—	—	—	—	—
1951	—	—	—	—	—	—	—
1952	—	—	—	—	—	—	—
1953	—	—	1	—	—	—	—
1954	—	—	1	—	—	—	—
1955	—	—	—	—	—	—	1 (4 in.)
1956	—	—	—	—	—	—	—
1957	—	—	1	—	—	—	—

7.6. The intensity of 1.75 in. per hour for areas round about Hyderabad has been selected after a study of a number of storms. All known storms from 1939 to 1957 are plotted in Fig. 22. The 'X' axis gives their duration at each time and their measured intensity is shown on the Y-axis. The number of points indicates the number of times that intensity has

occurred for a particular duration. This graph shows that the shorter the duration the greater the intensity. For areas round about Hyderabad, the value of the intensity for purposes of

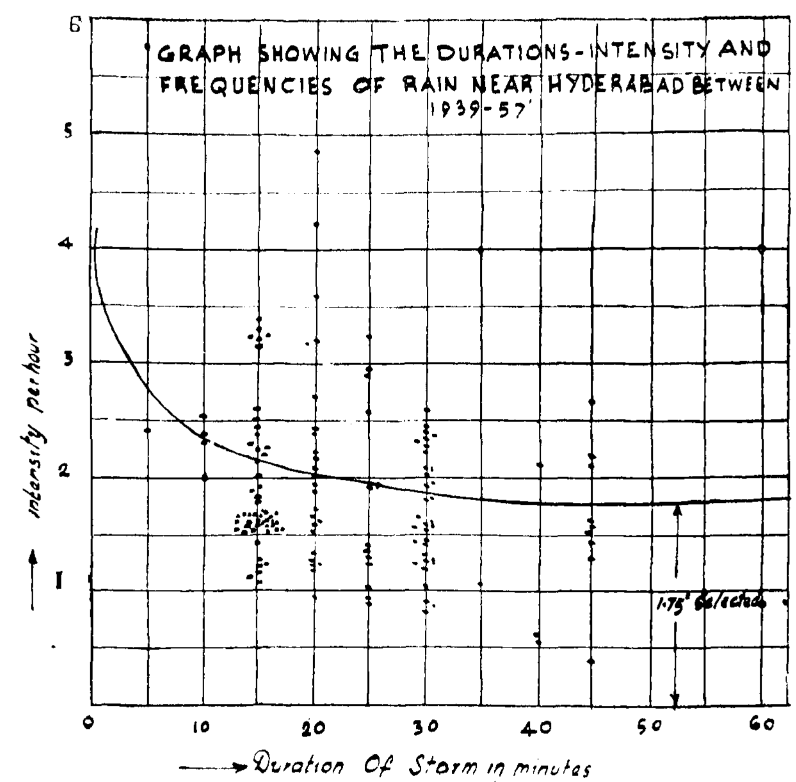


Fig. 22

design could be straightaway picked for any time of concentration of less than one hour duration, without having to calculate it by inserting the value of a in the equation (5). This graph also shows how stray are the durations of storms of more than half an hour.

7.7. The reliability of the data collected can be seen from a summary of the discharges obtained for streams, Table 22, and of examples worked out at the end for various places in Andhra Pradesh as compared to the discharges obtained and considered reliable from other aspects.

TABLE 22

Name of stream	By the rainfall data method given in the Paper	Estimated by other sources
1. Champavati (Example 1)...	1,70,000 cusecs	Not reliable
2. Kandaleru (Example 2) ...	39,000 ,,	35,000 cusecs
3. At Kadam (Example 3) ...	5,03,512 ,,	5,19,080 ,,
4. Bridge at mile 474/8 of N. H. 5 (Example 4)	5,850 ,,	5,056 ,,
5. Vakkeru Vagu (Example 5)	14,300 ,,	9,400 ,, originally assumed found inadequate, water bye-passed the bridge
6. Tajangigedda (Example 6)	20,600 ,,	14,800 to 17,470 cusecs

7.8. In areas where there are heavy forests, the water is held in three dimensions, i.e., on the ground and in the leaves of trees in depth. The effect of this is to make the flood even out and prolonged. The effect of intensity is thus interferred with. In such cases the effect of intensity may be considered to be 50 per cent effective only for purposes of estimating the runoff. This, however, requires more study and is yet to be proved.

7.9. Each of the factors mentioned in this Paper needs more detailed study and can form a field of investigation on its own merits.

ACKNOWLEDGMENT

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EXAMPLES

Example I

Construction of a bridge across Champavati on Vijayanagaram Palakonda Road at mile 5/7.

The river flows for about 19 miles in hills and 23 miles in plains. There is a bridge on National Highway No. 5 a few miles downstream having a catchment area of about 40.59 sq. miles more with a discharge calculated equal to 2,32,000 cusecs under that bridge.

Area of the catchment = 490 sq. miles.

1. Discharge calculated according to Lillie's formula = 1,50,000 cusecs.
2. Discharge calculated according to area velocity method, 2,55,600 to 3,03,100 cusecs. Here it was found that the H.F.L. line is difficult to fix because of a railway bridge immediately downstream and may be vitiated due to afflux and not reliable. This explains the wide variations and exaggerated discharge figures obtained by area velocity method.
3. Discharge according to Inglis formula = 1,54,000 cusecs.
4. Ryves formula = 66,000 cusecs.
5. According to Dickens assuming a value of C = 1600 = 1,66,000 cusecs.
6. According to Dickens assuming a value of C = 1170 = 1,21,400 cusecs

Discharge according to rainfall data collected :

1. Expected intensity on the catchment = 2.5 inch. per hour
2. Hilly portion

   Length = 19 miles
   Fall = 3200 feet
3. **Plains:**

   Length = 23 miles
   Fall = 320 feet

4. Area factor = $f(A)$ = 0.95
5. Value of a = 1
6. Coef. of discharage K = 0.95 (8.05 inches in a day)
7. Value of ϕ in rocky area = 11
   ϕ in plains = 12
8. Time of concentration

(1) **Time of concentration if worked in parts :**

The time of concentration at the end of hill with steep slope

(a) $$= \left[11 \times \frac{19^3}{3200}\right]^{\frac{1}{3}} = 2.85 \text{ hours.}$$

(b) when flat slope is produced for the above section

$$T_c = \left[11 \times \frac{19^3}{264}\right]^{\frac{1}{3}} = 6.70 \text{ hours.}$$

Nett hours gained = 6.70 − 2.85 = 3.85 hours.

(c) Time of concentration for the whole length taking the flat slope

$$T_c = \left[11.5 \times \frac{42^3}{584}\right]^{\frac{1}{3}} = 11.4 \text{ hours.}$$

Therefore the time of concentration

$$= 11.4 - 3.85 = 7.55 \text{ hours...(i)}$$

(2) **If the slope was taken to be uniform throughout then:**

$$T_c = \left[11.5 \times \frac{42^3}{3520}\right]^{\frac{1}{3}} = 6.6 \text{ hours...(ii)}$$

which will give an exclusive discharge as the time of concentration is less when compared with the T_c calculated according to change of slope of the catchment.

9. Average intensity on the catchment

$$T_c = \left[\frac{1+1}{7.55+1}\right] \times 2.5 = 0.60 \text{ inches.}$$

10. Then the maximum discharge that could be expected at this site will be

$$= 0.95 \times 490 \times 640 \times 0.95 \times 0.60$$
$$= 1{,}70{,}000 \text{ cusecs approximately}$$

which appears to be reasonable as compared to the reliable discharge measured at the road bridge on National Highway downstream, under extraodinary floods.

Example 2

Kandaleru is a stream at M.87/7 on Madras-Calcutta road (National Highway No. 5.) near Gudur. Most of the streams in this area are subjected to rolling floods and the assessment of their flood discharge is difficult from conventional methods. The discharge in the case of the stream referred to above has been calculated from (1) Ryves (28,000 cusecs), (2) Area Cross Section (33,602 cusecs) and (3) from gauging of the flood at the railway bridge (35,000 cusecs) about half a mile upstream. No other streams are joining this stream in that reach and so this could be considered as a reliable figure of discharge.

The particulars of the Kandaleru as affecting the site are given below:

A. (1) Catchment area = 320 sq. miles.
i.e. = 320×640 acres.

(2) Length of stream as measured from the topo sheets = 48 miles.

(3) Total fall of the stream in this distance = 420 feet. (Fall due to hills in the beginning has been ignored as it is for a short length and will not influence the general velocity of flow).

(4) Nature of gound: sandy: Clay as inspected at site and rocky at its beginning (1/3 distance).

B. (1) As most of the catchment area (shown 2 in Plate I) extends over the region of 2 in. per hour intensity and some in 2.5 in. per hour zone, I is taken to be 2 in. per hour as it is mostly in 2 in. zone.

(2) For this intensity the value of (a) in equation (4) from Fig. 9 is 1.00

(3) The ground being sandy clay and rocky the value of ϕ in equation 9 from Table 14 is 14.

The time of concentration T_c (eqn. 9)

for this catchment will then be $= \left[14 \times \frac{48^3}{420}\right]^{\frac{1}{3}}$
$= 16$ hours.

Using this value to find the intensity of rainfall effective for this catchment:

$$I' = \frac{1+1.0}{16.000+1.0} \times 2 = \frac{2 \times 2.0}{17} = 0.235 \text{ in. per hour}$$

As the rainfall per day is about 5 to 8 inches the value of (k) is taken to be 0.85 as the intensity is 2 in. per hour and mostly in that zone. The area factor from Fig. 5 is taken to be 0.95 for Rayalaseema. Then discharge $Q = 0.85 \times 320 \times 640 \times 0.95 \times 0.235 = 39{,}000$ cusecs which appears to agree with the discharge obtained at the Railway bridge site which is considered more reliable than others.

Example 3

Probable discharge from the catchment for the road arch bridge downstream of Kadam dam.

The top of the arch bridge was washed away during the recent heavy floods of 1958. (Photos 3, 5 & 6).

1. Catchment area = 1000 sq. miles.
2. Distance of the farthest point = 46 miles.
3. Fall = 1810 − 610 = 1200 ft
4. Type of soil = black cotton.
5. Intensity of rainfall = $2\frac{1}{2}$ in. per hour.

The discharge based on *one hour* and *one and half hour duration* of this intensity has been worked out considering that the catchment is likely to have a 12 in. downpour in a day. The latter will give the highest flood, the other will give a high flood.

6. Value of $\phi = 3$ for this black cotton soil & intensity.

Time of concentration $= \left[3 \times \frac{46^3}{1200}\right]^{\frac{1}{3}} = 6.2$ hours.

(1) Discharge based on one hour intensity duration:

Average intensity of rain on the catchment

$$= \frac{1+1}{6.2+1} \times 2.5 = 0.70 \text{ in.}$$

(a) Value of k = .95 for this intensity.

(b) Area factor considering the storm is of the same type as 12 in. rainfall = 0.95

$$Q = 0.95 \times 1000 \times 640 \times 0.95 \times 0.7 = 4,05,000 \text{ cusecs.}$$

(2) **Discharge based on $1\frac{1}{2}$ hour duration of this intensity as the rainfall is of the order of 12 in. and more per day:**

$$\text{Average intensity} = 2/3 \left[\frac{1.5+1}{6.2+1} \times 3.75\right]$$

$= 0.87$ inches.

$Q = 0.95 \times 1000 \times 640 \times 0.95 \times 0.87$

$= 5,03,512$ cusecs which closely approximates the estimated highest flood of 5,19,000 cusecs observed during 1958.

Example 4

A three span arch bridge at mile 474/8 on National Highway No. 5 (Madras—Calcutta Road) collapsed due to flood overflowing it on October, 1958. The probable flood at this place taking into account the rain that fell during that period as compared to the discharge worked on the H. F. L. mark on that day is given below:

Discharge as calculated by the area velocity method.

Cross section	Discharge cusecs
1200 ft upstream	5056
800 ft ,,	3715
100 ft ,,	4504

Rainfall method :

1. Catchment area = 8.7 sq. miles.
2. Length of the farthest point = 4 miles.
3. Fall in feet = 70
4. Type of soil = Sandy silt.
5. Intensity of rainfall = 2.45 inches per hour.

6. ϕ adopted for the above data = 20

$$Av. \quad t_c = \left[20 \times \frac{4^3}{70}\right]^{1/3} = 2.66 \text{ hours.}$$

Average effective intensity over the catchment

$$= \frac{1+1}{2.66+1} \times 2.45 = 1.32 \text{ in.}$$

Area factor = 0.95 $\quad K = 0.85$

$Q = 0.85 \times 8.7 \times 640 \times 0.95 \times 1.32$

$= 5,850$ Cusecs.

This is supported by the discharge calculated from the H. F. L. actually observed during floods when the bridge collapsed.

Example 5

The Vakkeru Vagu on Kurnool diversion of National Highway 7 was bridged but the approaches to this were washed out due to the flood of 1958. The bridge was designed for 9,400 cusecs taken previously based on Ryves formula. The discharge for rainfall data is worked out below:

1. Area of catchment = 70 sq. miles
2. Length of stream = 20 miles
3. Fall of stream = 277 feet
4. $\phi = \dfrac{10.02+19.00}{2} = 14.5$
5. Intensity of rainfall = 1.75 in. per hour
6. Time of concentration $= \left[14.5 \times \dfrac{20.4^3}{277}\right]^{1/3}$

$= 7.6$ hours.

Average intensity of rain that may be expected on the catchment $= \dfrac{1+1}{7.6+1} \times 1.75 = 0.405$ in.

Area factor = 0.99 $\quad K = 0.8$

$\therefore \quad Q = 0.8 \times 70 \times 640 \times 0.99 \times 0.405 = 14,300$ cusecs.

which shows the inadequacy of the waterway and confirms the necessity for working out the discharge on a more rational basis.

Example 6

Tajangigedda is a stream in Chintapalli area of the Agency. The proposal of a bridge at M. 2/8 of Lamsingi to Tajangi road has been investigated. The discharge as computed according to area velocity and Dickens method works out as follows:

	Discharge (cusecs)
1. Dickens	14,820
2. 300 ft U/s	16,550
3. At site of bridge	17,180
4. 300 ft D/s	17,470

Area of catchment 19.45 sq. miles

1. **Hill portion**

Length = 1.7 miles
Fall = 880 ft

2. **Plains B. C. soil**

Length = 5.7 miles
Fall = 244 feet

T_c in hill with steep slope $= \left[14 \times \frac{1.7^3}{880}\right]^{1/3} = 0.43$ hours

T_c in hill with gentle fall continued $= \left[14 \times \frac{1.7^3}{72}\right]^{1/3} = 0.85$ hours

Nett gain = 0.42 hours

T_c with flat slope for the full length $= \left[6 \times \frac{7.4^3}{316}\right]^{1/3} = 2$ hours

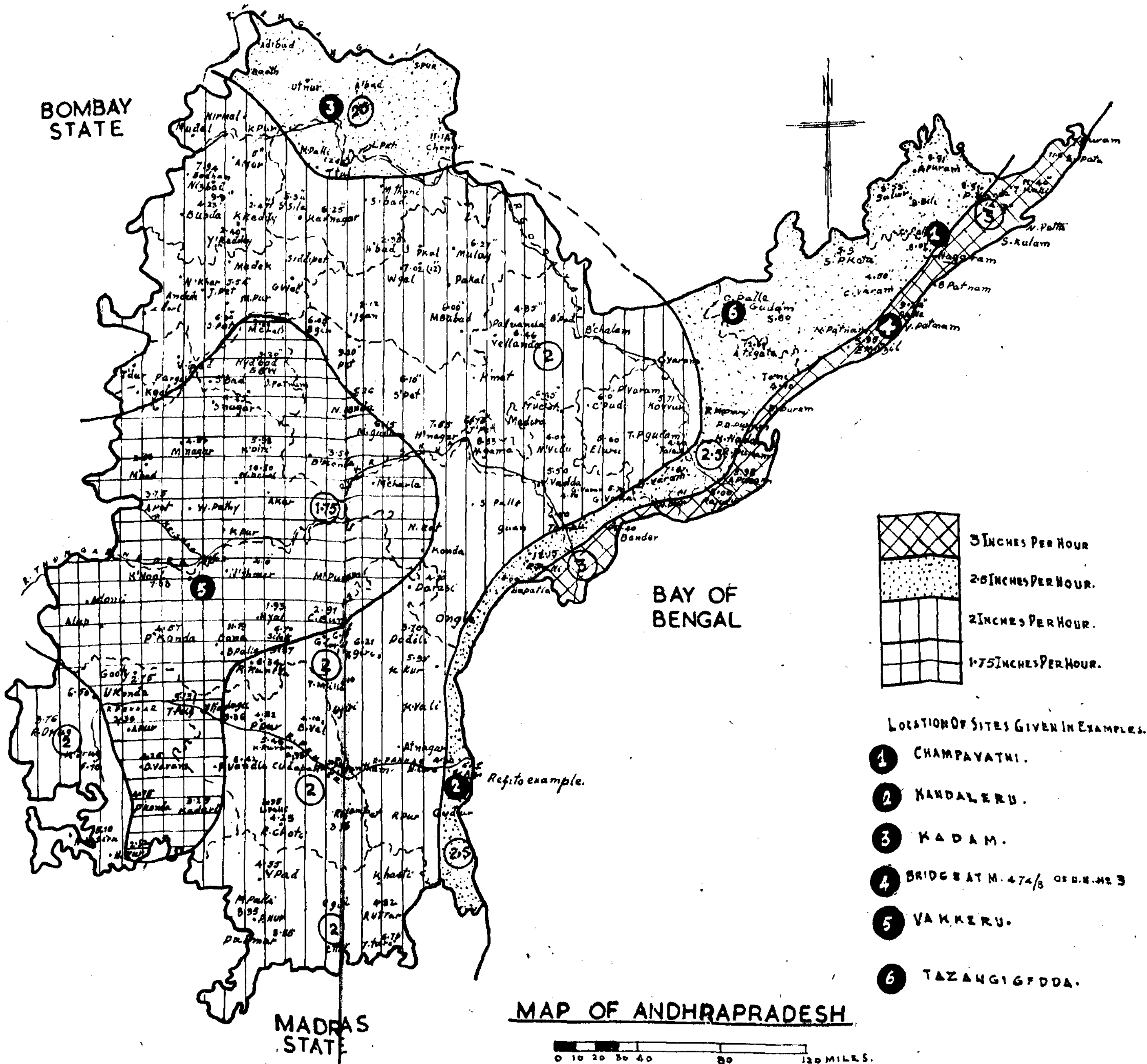
Nett $T_c = 2.0 - 0.42 = 1.48$ hours

Value of $\alpha = 1$ as T_c is > 1.

Average intensity $= \frac{1+1}{1.48+1} \times 2.5 = \frac{5}{2.48} = 2$ in.

$Q = 0.85 \times 19.45 \times 640 \times 0.98 \times 2$
$= 20{,}600$ cusecs.

PLATE I



HIGHEST RAINFALL IN A DAY SO FAR RECORDED AND ZONES SHOWING HOURLY INTENSITIES PROPOSED

PLATE II

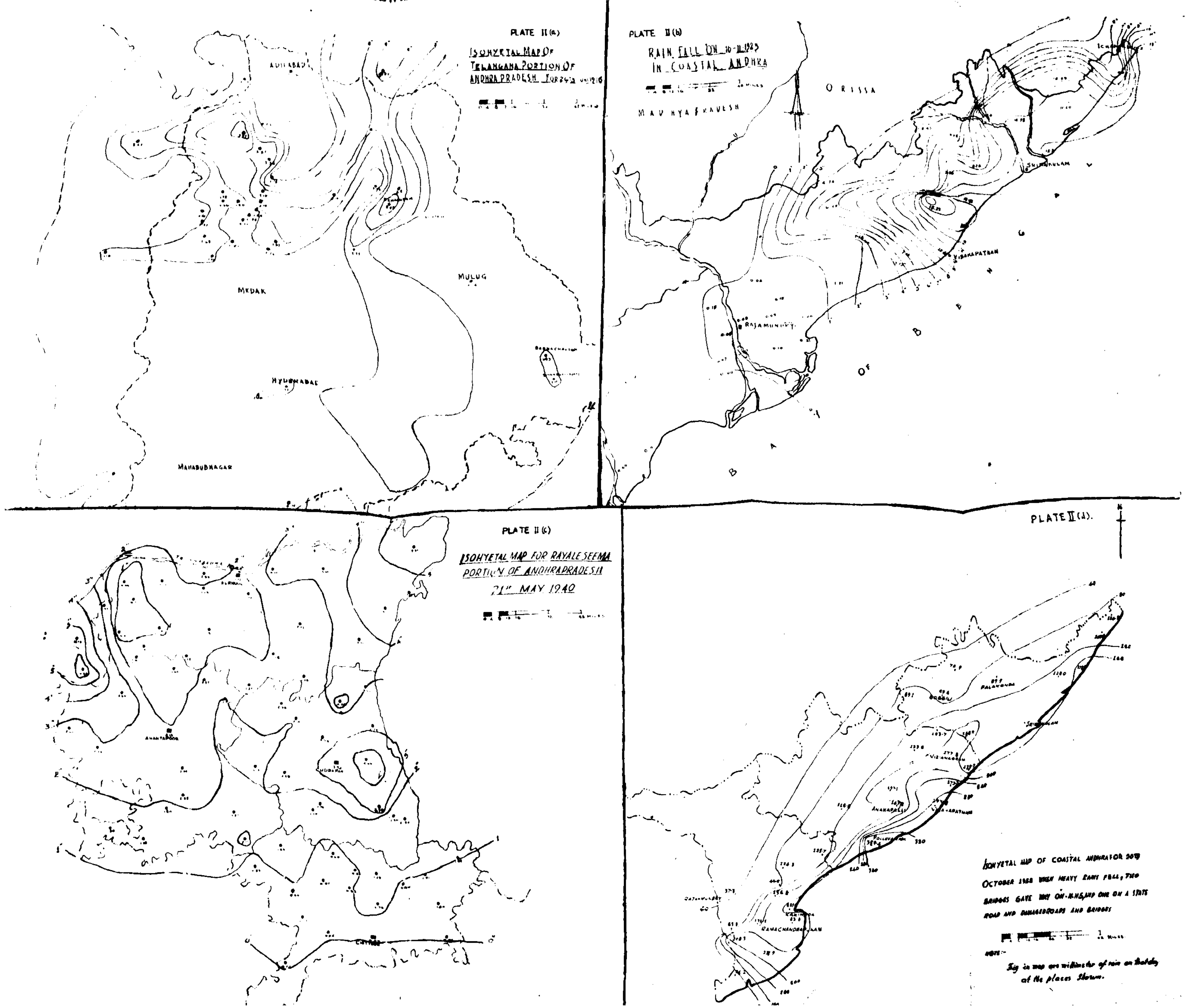
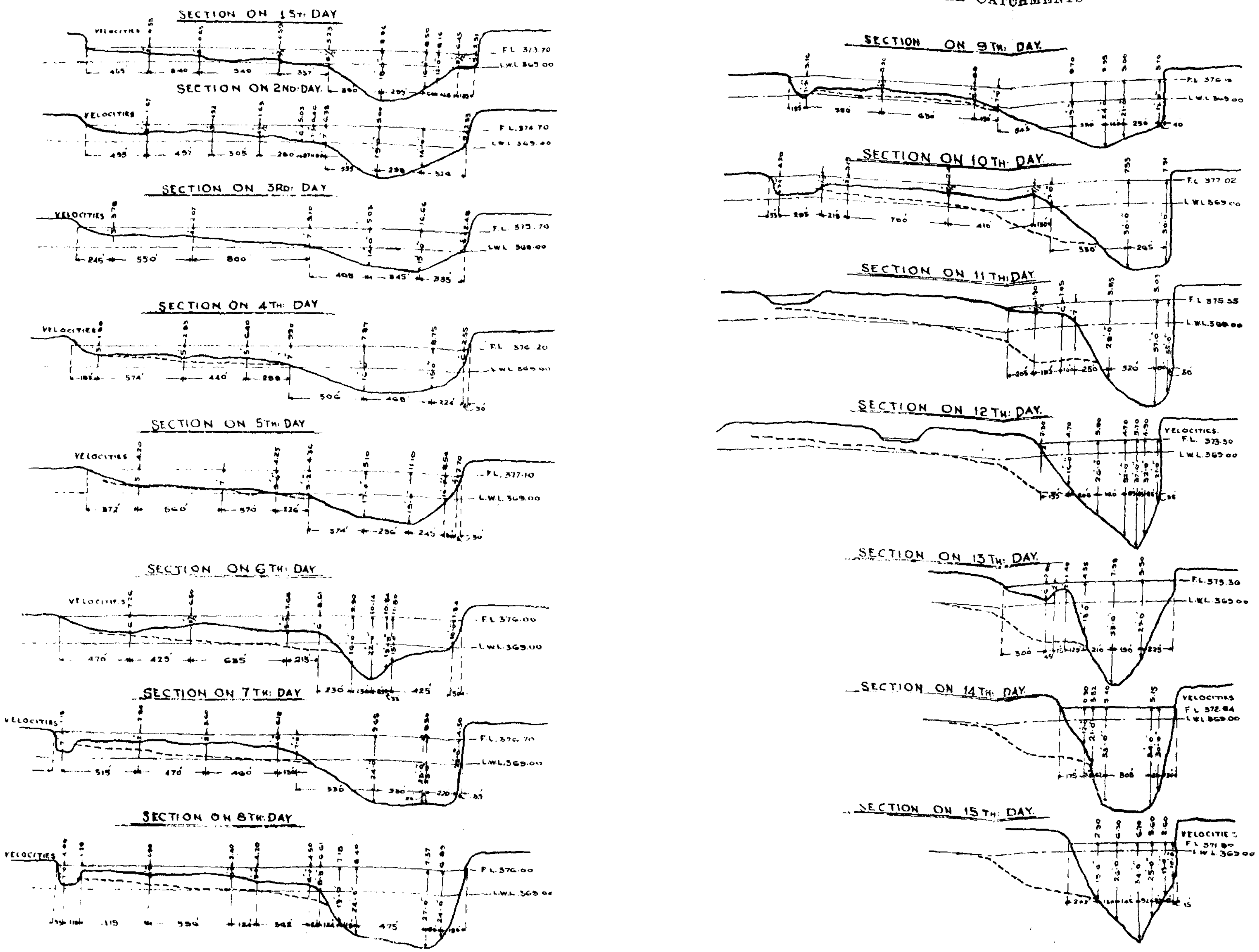


PLATE II (a)
ISOHYETAL MAP OF TELANGANA PORTION OF ANDHRA PRADESH
MEDAK
MULUG
HYDERABAD
MAHABUBNAGAR
PLATE II (b)
RAIN FALL ON 10-11-1923 IN COASTAL ANDHRA
ORISSA
RAJAMUNDRY
PLATE II (c)
ISOHYETAL MAP FOR RAYALESEEMA PORTION OF ANDHRAPRADESH
21st MAY 1940
ANANTAPUR
PLATE II (d)
NOTE:-

PLATE III



SECTIONS OF RIVER SUTLEJ TAKEN DURING FLOODS OF 1872

SCALE:- HOR: 0 250 500 1000 FEET.
VER: 0 15 30 60 FEET.

Photo 1.
FULL VIEW OF DIMNI BRIDGE

Paper No. 217

"DESIGN AND CONSTRUCTION OF PRESTRESSED CONCRETE BRIDGES ACROSS KUNWARY RIVER"

By

Y. K. PATIL,† B.E. (CIVIL) AND C.V. KAND, *B.E. (CIVIL).

CONTENTS

SYNOPSIS

The River Kunwary flows through the northern districts of Madhya Pradesh (Bhind and Morena). The river crosses the Morena—Ambah—Mehgawan Road near Dimni and Budhara while it crosses the Bhind—Etawah Road at Didi. The former is an important feeder road connecting two National Highways viz., Agra-Bombay Road and Gwalior-Etawah Road (proposed National Highway).

At these three crossings high level prestressed concrete bridges are being constructed. Bridges at Didi and Dimni have a maximum span of 132 ft centre to centre with the R.C.C. deck slab supported on four main girders while at Budhara there are three main prestressed girders having a maximum span of 127 ft centre to centre. The girders are cast in-situ and post-tensioned by Freyssinet's method. The supporting piers are of R.C.C. shell-type and rest upon well foundations.

The Paper deals with the hydraulic investigations, design and construction of these bridges.

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1. INTRODUCTION

1.1. The Kunwary river flows eastwards through the districts of Bhind and Morena in the northern region of Madhya Pradesh. This river is a tributary of the Chambal which forms the boundary line of Madhya Pradesh, Rajasthan and Uttar Pradesh, Plate I. The Kunwary crosses Agra—Bombay National Highway at a distance of three miles from Morena which is district headquarters and a central marketing place for the agricultural produce in the region. There is a high level bridge at this crossing but at other crossings on internal district roads there are only 'Rapats' (causeways). At Dimni, Budhara and Didi crossings, the traffic was ferried across the river during monsoon. This arrangement was both inadequate and cumbersome and the growth of vehicular traffic in these regions justified the construction of three high level bridges in the Second Five-year Plan.

1.2. The course of the Kunwary is mostly over a terrain of yellow sandy soil of low cohesive property in its wet condition which has caused soil erosion resulting in deep ravines in considerable portion of its catchment area. The rainfall is characteristic in being sudden and heavy and as a result of that the river has gorged a deep bed along its course and is fordable at very few points.

Such uncommunicable topography has naturally been very inviting to the Thugs and Pindharies of the past and the notorious dacoit gangs of present times to favour this ravinous region for their cryptic existence where the hands of law cannot reach easily. Even in a direct encounter the armed forces of the Government are very much at disadvantage and it is very unlikely that mere show of arms could ever wipe out the dacoit menace from this area.

The socio-economic analysis of this region shows that unless poverty in this area is removed, the criminal outlook of the inhabitants may not go and the prime measure adopted by Government towards this end is to give maximum impetus to agriculture. The so-called sandy soil has proved to be very fertile if properly irrigated and manured. The canal water from the Chambal and the soil-erosion control programme would entirely change the face of the region by improvement in the agricultural potential in the near future. This in its turn will need more roads and better means of communication. The construction of bridges at Dimni, Budhara and Didi is the beginning of a new epoch to this ravaged region.

1.3. Dimni, Budhara and Didi are all prestressed concrete bridges and their locations are shown on the index map, Plate I. Dimni bridge is on Morena—Ambah—Road at a distance of twelve miles from Morena while Budhara is thirty five miles away on the same route from Morena, connecting it to Bhind via Ambah and Mehgawan. The third bridge near Didi is five miles from Bhind on the Bhind—Etawah—Road which is likely to be upgraded as a National Highway. Dimni bridge was opened to traffic in January 1959. The construction of Didi bridge is complete and it has been opened to traffic recently. At Budhara, foundations, substructure and 75 per cent of superstructure are ready. The remaining work would be completed after the monsoon of 1959. Thus all these bridges would be opened to traffic before the end of 1959.

2. HYDRAULIC INVESTIGATIONS

Hydraulic data, design flood discharge and design depth of foundations are given in Table 1. The investigations are based on the following principles.

2.1. Design Flood Discharge

The maximum flood discharge at bridge site is likely to occur only when the catchment area is saturated by earlier rains. The discharge is influenced by—

(i) nature and expanse of catchment area,

(ii) extent, intensity and duration of rainfall, and

(iii) time of concentration of the flood.

When the maximum discharge occurs, other factors such as temperature, surface detention, etc., have least significance. The magnitude of this discharge is calculated by various methods. At the site of Dimni bridge, the following figures of maximum discharge were obtained:

DATA AT DIMNI BRIDGE SITE

Catchment area	...	$A = 1{,}140$ sq. miles
Length of river	...	$L = 106$ miles
Particulars of cross section at 700 feet upstream of the site:		
Waterway at H.F.L. R.L. 540.00	...	$A_w = 19{,}100$ sq. ft.
Wetted perimeter	...	$P = 1{,}300$ ft.

Hydraulic mean depth $\left(\frac{A_w}{P}\right)$ ... ... $r = 14.54$ ft.

Bed slope ... ... $s = 1$ in 5,000

Coefficient of rugosity ... ... $n = 0.025$

(a) Discharge by Inglis' formula $Q = \frac{7000\,A}{\sqrt{A+4}}$ ... $Q = 2{,}41{,}000$ cusecs

(b) Discharge by Dicken's formula (assuming $C = 800$) $Q = CA^{\frac{3}{4}}$ ... $Q = 1{,}60{,}000$ cusecs

(c) Discharge by area velocity method using Kutter's velocity $Q = A_w V_{ku}$ ... $Q = 1{,}03{,}000$ cusecs

$$V_{ku} = \frac{41.6 + \frac{1.811}{n} + \frac{0.00281}{s}}{1 + \left(41.6 + \frac{0.00281}{s}\right)\frac{n}{\sqrt{r}}} \times \sqrt{rs}.$$

The empirical formula developed by Inglis is more suitable for fan catchments while Dickens formula requires very careful guess for adopting suitable coefficient, depending upon various unknown factors. In this region value of C is taken as 800 for big catchments. Inglis and Dicken's discharges are related to catchment area only whereas in the 'Area velocity method' velocity of the flow by Kutter's formula and the cross sectional area at H.F.L., have been considered.

(d) **Rainfall data method**

The discharges obtained in (a), (b) and (c) cannot be relied upon because important factors such as actual rainfall and shape of catchment have not been accounted for. These factors are considered in the "rainfall data method" based on the Paper* read by the Author before the Research Committee of the Central Board of Irrigation in the year 1945.

In this Paper, the Author worked out the time of concentration of flood as:

$$T = \frac{1.34\,A^{\frac{1}{3}}}{F^{1/3} \times P^{1/6}} \qquad \text{......(i)}$$

where T = Time in hours
A = Catchment area in sq. miles
F = Silt factor
P = Intensity of rain per hour during the period "T"

* "Flood Discharges of Rivers in India" by Y. K. Patil

In working out formula (i), fan catchment having length of river equal to $1.41\sqrt{A}$ was assumed. Substituting actual length (L), the time of concentration is given by formula (ii) which can be applied to any shape of catchment.

$$T = \frac{0.95\,L}{A^{1/6} \times P^{1/6}} \cdots (\text{for } F = 1) \qquad \text{......(ii)}$$

Rainfall data for the Kunwary catchment

Maximum intensities of rainfall observed in the northern districts of Madhya Pradesh are as follows:

Gwalior district	5.70 in. in 14 hours	$P = 0.407$ in.
Guna district	8.82 in. in 16 hours	$P = 0.551$ in.
Shivpuri district	8.10 in. in 14 hours	$P = 0.579$ in.

For the adjoining districts of Bhind and Morena where the climatic conditions are more or less the same, the maximum rainfall intensity is assumed to be 8.10 in. in 14 hours ($P = 0.579$ in.).

The time of concentration of flood (T) is calculated by successsive approximations.

First approximation with P equal to maximum intensity of rainfall in this region (0.579 in.)

$$T_1 = \frac{0.95 \times 106}{1140^{1/6} \times 0.579^{1/6}}$$

L = 106 miles
A = 1140 sq. miles
P = 0.579 in. during a period of 14 hours

T_1 = 34 hours

But to get exact time of concentration, P for a period 'T_1' has to be considered. Thus —

$$P = \frac{8.10}{34} = 0.2382 \text{ in.}$$

Second approximation with $P = 0.2382$, period of concentration works out to:

$$T_2 = \frac{0.95 \times 106}{1140^{1/6} \times 0.2382^{1/6}}$$

$= 39.5$ hours

$\therefore$ Reduced intensity during T_2

$$P = \frac{8.1}{39.5} = 0.205 \text{ in. per hour.}$$

The intensity of runoff which is always less than the intensity of rain is given by:

$$P_r = 0.96 \times P^{1.02}$$
$$= 0.96 \times 0.205^{1.02}$$
$$= 0.1895 \text{ in. per hour}$$

$\therefore$ Discharge per square mile

$$q = 640 \times 0.1895 = 121.4 \text{ cusecs}$$
$$\therefore Q = 1140 \times 121.4$$
$$= 1,38,500 \text{ cusecs.}$$

This value is adopted as the design discharge for Dimni bridge. Design discharges for Budhara and Didi, given in Table 1, are likewise calculated.

It must be emphasised that this particular catchment area is elongated. Average width up to Dimni is $\frac{1140}{106} = 10.8$ miles, and length to breadth ratio is 9.8. Had this catchment area been fan-shaped, the length of the river would have been much less than 106 miles. Consequently, the time of concentration would have been lesser resulting in more intensity of runoff than the above figures.

2.2. Design Depth of Foundation

The depth of foundation, firstly, depends upon the safe bearing capacity of strata met with in foundations and, secondly, on the considerations of scour depth. Where there is no rock in the foundations and the strata is erodible, scour depth is the guiding factor. At Dimni, borings were taken upto 60 feet below the lowest bed level. Results in Table 1-B reveal that no rock was met within 60 ft. The strata at this depth was yellow soil mixed with kankar.

According to Clause 109 of IRC Code, normal depth of scour $= \frac{0.473 Q^{1/3}}{f^{1/3}}$. Maximum scour depth at the nose of the pier is double the normal scour depth. The Code further lays down that the minimum depth of foundation should be 4/3 times the maximum depth of scour. The scour depth formula is true for alluvial streams which have reached regime conditions. In this case the streams are quasi-alluvial, hence the formula is only

partially true. Experiments carried out at Poona Hydraulic Research Station reveal the following facts :*

(a) The depth of scour round a pier is little affected by the general material of bed so long as it is incoherent but where scour is resisted, as by a clay bed, scour round a pier may increase and will be further increased by the effect of curvature of flow which is normally induced in the vicinity of obstructions where there is resistant bed material.

(b) Where a sudden increase of discharge occurs before the bed has time to scour or where there is resistant bed layer, scour action is likely to be considerably intensified.

(c) Once the bed becomes shrouded by super-charge of sand flowing near the bed, the scour hole round the pier tends to decrease in depth.

(d) In general, (*b*) & (*c*) will be in action at one and the same time except where curvature of flow side-tracks the bed sand. This means that the curvature of flow is the dominant factor causing maximum scour.

The formula based on these experiments for scour is $D = 1.7\, b^{0.22}\, q_c^{0.52}$, where b = width of pier and q_c is the discharge per rft. on the upstream of the pier. This formula generally holds independently of the grade of the bed sand.

In streams where there is cohesive bed material such as clay or clay mixed with kankar, it is obvious that the tendency to scour is reduced and there will be practically no scour in such strata.

The depths of foundations in such cases, therefore, need not be governed by the formula given in Clauses 109 and 110 of IRC Code for obvious reasons. The depth of foundation below the maximum scour depth is governed by the adequacy of anchorages developed by passive pressure of the surrounding earth, to withstand the overturning forces and moments and the safe bearing capacity of the soil. It has been empirically found that this depth (below maximum scour) is not more than 4 feet

* Research Publication No. 6 of C. I. & H. R. Station, Poona, 1941-42; pages 32-33.

for non-arch type of bridges. It depends upon the span of the bridge and depth of piers. Clause 110, therefore, recommends a minimum depth of 4 feet below the maximum scour level for founding the piers.

The bed stratas at Dimni, Budhara and Didi are neither fully sandy nor fully clayey. The top layers upto 20 ft contain soil with large percentage of sand, the proportion of which goes on decreasing at lower depths. However, maximum scour depth was calculated by assuming alluvial stream. Design foundation depth was based on this maximum scour depth but actual depth for plugging the wells was determined by observing the resistance offered by the soil during sinking of wells.

TABLE 1-A

Hydraulic data

S. No.	Details	Dimni bridge	Budhara bridge	Didi bridge
1.	Catchment area (in sq. miles)	1140	2030	2400
2.	Length of river up to the bridge (in miles)	106	128	158
3.	Design discharge (in cusecs)	1,38,500	2,21,800	2,39,500
4.	Linear waterway provided (feet)	376	479	500
5.	Formation level ...	R. L. 552.58	R. L. 129.00	R. L. 516.00
6.	High flood level ...	R. L. 540.00	R. L. 116.00	R. L. 504.00
7.	Low water level ...	R. L. 502.50	R. L. 84.50	R. L. 470.00
8.	Lowest scour level (calculated)	R. L. 491.00	R. L. 59.00	R. L. 445.00
9.	Design foundation level	R. L. 474.00	R. L. 40.00	R. L. 425.50
10.	Actual lowest foundation level	R. L. 465.00	R. L. 32.00	R. L. 423.62
11.	Average height of bridge between formation and foundation level (feet)	83	85	85

Note: The datum for reduced levels is different in each case.

TABLE 1-B

Bore data at Dimni bridge site

Reduced level	Depth below bed level	Depth of the strata	Nature of soil
	ft in.	ft in.	
511.77	0 0	—	Ground level
501.21	10 6	10 6	Sand mixed with soil
494.71	17 0	6 6	Sand
489.71	22 0	5 0	Blackish soil with sand
479.71	32 0	10 0	Yellow Padua soil
469.71	42 0	10 0	Yellow Padua soil with little kankar
448.71	63 0	21 0	Yellow Padua soil mixed with kankar

3. TYPE AND DETAILS OF BRIDGES

3.1. Type

Table 1-A reveals that the average height between the formation level and the lowest level of foundation is approximately 85 ft out of which 40 ft are below the bed of river in a sandy strata. In such a case, well foundations are unavoidable. Economic span for these conditions would be in the vicinity of 120 ft to 140 ft. Superstructure may be either balanced cantilever R.C.C. T-beam and slab or prestressed concrete girders and R.C.C. deck slab. The State P.W.D. proposed R.C.C. superstructure on masonry piers. However, among the tenders received by the department, the tenders of Messrs Gammons with R.C.C. piers and prestressed concrete superstructure, being the lowest, were accepted. Relative economy in the consumption of cement and steel in the prestressed concrete design over the R.C.C. alternative is discussed under Statistics (Section 8).

3.2. Details

The foundations, piers and abutments of all the bridges are similar with slight variation in the heights. The superstructures of Dimni and Didi bridges are identical while at Budhara bridge, three girder system is adopted in place of the four girder system of Dimni and Didi.

Table 2-A gives the spans which are actually provided.

TABLE 2-A

Spans and lengths of bridges

S. No.	Bridge at	Number of spans	Span C/C of piers	Length of super-structure	Total length including return walls
			ft in.	ft in.	ft in.
1.	Dimni ...	3	130 1 132 0 130 1	392 2	446 2
2.	Budhara ...	4	125 0 127 0 127 0 125 0	504 0	554 0
3.	Didi ...	4	130 1 132 0 132 0 130 1	524 2	578 2

Details of bridges are given in Table 2-B.

TABLE 2-B

Salient features of bridges

Particulars	Didi & Dimni bridges Four girder system span 132 ft-2 in.	Budhara bridge Three girder system span 127 ft-0 in.
(Note—Plate II gives general details of Dimni bridge)		
1. FOUNDATION—colcrete (1 : 3) twin wells		
(a) Below piers (27 ft – 9 in. long)	Outer diameter – 15 ft – 0 in. Inner ,, – 10 ft – 6 in.	Refer Plate III.
(b) Below abutments (29 ft – 9 in. long)	Outer ,, – 16 ft – 0 in. Inner ,, – 11 ft – 6 in.	Refer Plate IV.
2. WELL CAPS—R. C. C. 1 : 2 : 4 high grade mix		
(a) Below piers	27 ft – 9 in. × 15 ft – 0 in. × 24 in. thick	
(b) Below abutments	29 ft – 9 in. × 18 ft – 0 in. × 30 in. ,,	
3. PIERS—R. C. C. 1 : 2 : 4 high grade mix, cellular type (Plate III)		
Overall size	26 ft – 6 in.×6 ft – 6 in. (Semi-circular cut and ease water)	24 ft – 2 in.×6 ft 8 in.
(a) Outer wall	12 in. thick	8 in. thick

TABLE 2-B *contd.*

Particulars	Didi & Dimni bridges Four girder system span 132 ft – 2 in.	Budhara Bridge Three girder system span 127 ft – 0 in.
(b) Cross walls (located below each main girder)	4 Nos. 6 in. thick @ 5 ft – 6 in. centre to centre	3 Nos. 6 in. thick @ 8 ft – 0 in. c/c.
4. PIER CAP—R. C. C. 1 : 2 : 4 high grade mix		1 ft – 6 in. thick (Plate III).
	29 ft – 0 in.×9 ft – 0 in	26 ft – 6 in.×9 ft 0 in. (Semicircular ends)
5. BEARINGS—R. C. C. 1 : 1 : 2 mix special grade		(Plate V)
	18 ft – 6 in.×0 ft – 9 in.×1 ft – 6 in.	17 ft – 9¼ in.×0 ft – 9 in.×1 ft – 6 in.
6. MAIN GIRDERS—special grade concrete 1 : 1½ : 3 (prestressed)		
(a) Overall depth	6 ft – 10 in.	7 ft – 0 in.
(b) Top flange (deck-slab 23 ft wide)	0 ft – 6 in. thick	0 ft – 7 in. thick
(c) Bottom flange	15½ in.×10 in.	16½ in. × 9 in.
(d) Thickness of web.		(Plates VI & VII)
at the centre	5 in.	5½ in.
at the support	15½ in.	16½ in.
(e) Diaphragms	6 in.×5 ft – 6 in. @ 15 ft – 7½ in. c/c.	6 in.×5 ft – 7½ in. @ 20 ft – 1¼ in. centre to centre
(f) Cables	74 Nos. 19 each in outer beam and 18 each in inner beam	68 Nos. 23 each in outer beam and 22 in inner beam
Cables projecting out from deck slab	14 Nos.	16 Nos.
(g) Spacing of main girders.	5 ft – 6 in.	8 ft – 0 in.
(h) Clear roadway	22 ft – 0 in. with 9 in. kerbs on each side	
7. PARAPETS	Pipe railing (3 tier) carried through precast R. C. C. posts spaced at 6 ft c/c.	
8. ABUTMENTS & RETURN WALLS	R. C. C. 1 : 2 : 4 high grade mix box type end abutment with front and return walls in beam and slab construction, the walls being stiffened by a centrally placed counterfort in the front wall and cross ties between the return walls......all of them anchored in well cap slab. A portion of return wall is cantilevered out beyond the rear face of well cap. (Refer Plate IV).	

4. SPECIFICATIONS AND PERMISSIBLE STRESSES

Brief specifications of various structural materials used in the construction work are as follows:

4.1. Concrete

Specifications of concrete mixes for various parts of the bridges are indicated in Table 2-B. Coarse aggregate for all R.C.C. work was obtained from quarries of black trap near Gwalior, while sand was obtained from Chambal or Sindh river as Kunwary sand is not found suitable for the purpose. The mixes were batched by volume and thoroughly vibrated in order to get the specified strengths. Ordinary Portland cement of ISI specifications was used.

4.2. Colcrete

From the view point of composition, colcrete is in between concrete and stone masonry. The process of casting is very quick. Rubble upto the maximum size of 9 in. is packed between formwork. Cement and sand in the ratio (1:3) are mixed to form a slurry. About 7 gallons of water are required per bag of cement. The slurry is carried through pipes and injected in the voids of packed rubble under pressure. The slurry fills the voids and a dense mass having a strength equivalent to 1:3:6 cement concrete is formed. The colcrete is used for well steinings and bottom plug. 100 cu.ft. of colcrete require about 10.5 bags of cement.

4.3 Steel

Tested mild steel was specified for R.C.C. works while high tensile steel having an ultimate strength of over 100 tons per sq. in. was used for prestressed girders.

4.4. Anchorages

The girders were prestressed by Freyssinet post-tensioning method. The male and female cones required for anchorages were imported from France.

4.5. Permissible Stresses in Concrete and Steel

Permissible stresses in concrete and steel, specified above, are as follows:

(A)	**Special grade concrete 1 : $1\frac{1}{2}$: 3 mix**	lb per sq. in.
(i)	Cube strength at 28 days (6 in. cubes)	5000
(ii)	Ultimate tensile strength	500
(iii)	Permissible permanent compressive stress	1667
(iv)	Permissible compressive stress during prestressing	2100
(v)	Permissible tensile stress	167
(vi)	Young's modulus F_c	5×10^6
(vii)	Modular ratio m	6
(B)	**High grade concrete 1 : 2 : 4 mix**	
(i)	28 days cube strength	3000
(ii)	Permissible compressive stress	950
(iii)	Shear or tensile stress	95
(iv)	Modular ratio $m =$	15
(C)	**Mild steel**	
(i)	Permissible tensile stress	18000
(ii)	Permissible compressive stress	18000
(iii)	Young's modulus E_s	30×10^6
(D)	**H. T. steel**	tons per sq. in.
(i)	Ultimate strength	100 to 110
(ii)	0.1 per cent proof stress	80
(iii)	Permissible stress	66
(iv)	Final prestress with 15 per cent losses	56
(v)	Young's modulus E_s	29/ 30×10^6 lb per sq. in.

5. DESIGN

It is not possible in the scope of this Paper to give detailed designs of all parts of the bridges. Important features only are discussed below. The bridges are designed for I.R.C. Class A, two train load. Maximum velocity of flood is assumed to be 13 ft per second for the design of piers and wells. The design is based on conventional elastic theory.

5.1. Foundation

(a) Safe load on foundation

Maximum depth of foundation below the lowest bed level is 37 feet, 50 feet and 44 feet for Dimni, Budhara and Didi bridges respectively. It has been pointed out that bed strata at this depth is yellow soil mixed with kankar. Safe load on foundation at a depth of, say, 37 feet, by assuming purely sandy bed and purely clayey bed, works out to:

First assumption : purely sandy strata

Assume water table up to the bed level

$$q = \tfrac{1}{3} \times \frac{r}{2 \times 2240} \left\{ \tfrac{1}{2} BN_r + D_f N_q - D_f \right\} \qquad \text{(i)}$$

where q = safe load in tons per square foot

B = the width of foundation in ft

r = unit weight in lb per cu. ft.

N_r = bearing capacity factor

N_q = -do-

N_r and N_q depend upon the angle of repose of the sand. Assuming $\phi = 34^\circ$, values of N_r and N_q are taken from 'Foundation Engineering' by Peck, Hanson and Tornburn, page 220:

N_r = 25

N_q = 25

B = 15 ft

D_f = 37 ft

r = 110 lb per cu. ft.

Substituting these values

q = 8.8 tons per sq. ft. ... (a)

Second assumption : fully clayey soil

(when $D_f = 0$; q = 1.5 tons per sq. ft. at the bed level)

Photo 2.

Traffic on Didi bridge

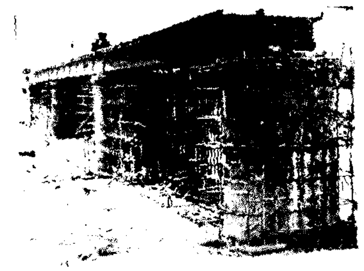


Photo 3.

Budhara bridge under construction

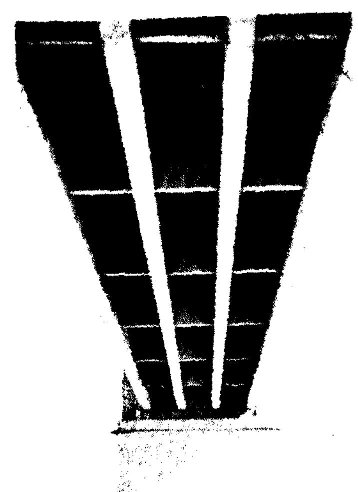


Photo 4.

Girder system of Didi & Dimni bridges, view of the underside

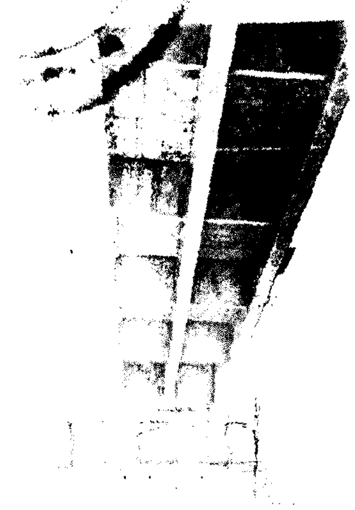


Photo 5.

Girder system of Budhara bridge

Photo 6.

Diver's equipment

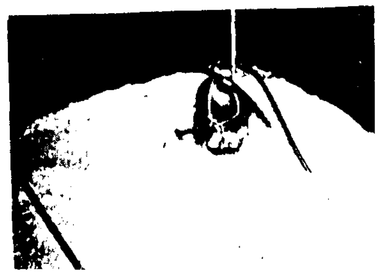


Photo 7.

Diving in the well

Photo 8.

Test load of Dimni bridge

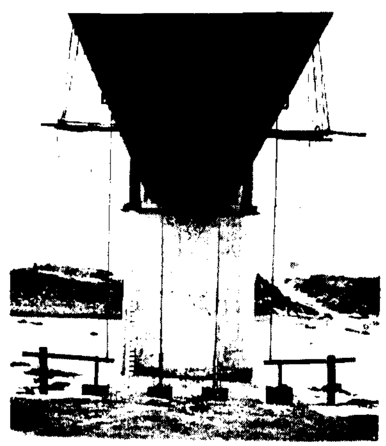


Photo 9.

Measuring of deflection on Dimni bridge

Assume compressive strength of the soil $q_u = 8$ tons per sq. ft.

$$q = 0.95 \times q_u \left(1 + 0.3 \frac{B}{L}\right) \quad \ldots \quad \text{(ii)}$$

B = width of pier = 15 ft .

L = length of pier = (27.75 – 1.75) = 26 ft

q = 9 tons per sq. ft. ... (b)

In formulae (i) and (ii), values of ϕ, r and q_u are to be found out by actual field experiments. No such experiments were carried out but safer values were assumed. These calculations are made only to have a rough idea about the safe bearing capacity of soil, assuming it was either purely sandy or purely clayey. Actual nature of soil being different, a safe capacity of *7 tons per sq. ft.* has been assumed in the design of foundations. While deciding this figure, foundations of existing bridges across the Kunwary and other rivers in this region were studied. Full load test on Dimni bridge and on the wells of Budhara bridge proved that the assumed bearing capacity of 7 tons per sq. ft. was quite safe.

(b) Well steining and well cap below piers

Design load and moments for wells below central piers of Dimni bridge are as follows:

(i)	Dead load from superstructure pier and wells deducting the effect of buoyancy	867 tons
(ii)	Live load	230 tons
(iii)	Moment in the direction of road	2120 ft tons
(iv)	Moment in the direction of current	1876 ft tons

Maximum pressure on steining is 12.75 tons per sq. ft. which can be safely borne by colcrete without reinforcement. However, 45 number of $\frac{3}{4}$ in. diameter vertical bars and $\frac{1}{2}$ in. dia. circular rings at 3 ft centre to centre were provided in the steining in order to withstand constructional stresses during sinking. The well cap below piers is designed as a freely supported circular slab with an effective diameter of 11 ft 6 in. Bending moments thus obtained are reduced by 20 per cent to allow for the effect of partial fixity. In the case of wells supporting the end abutment and box return walls, direct load is 1305 tons. The moments in both the directions are much less than those for central wells. However, the diameter of end wells is increased by 1 foot (11 ft 6 in.) in order to accommodate the abutment and return walls properly.

5.2. R.C.C. Cellular Piers—Dimni Bridge

Maximum height of pier shaft between the well cap and the pier cap is 38 ft 0 in. It is designed for a direct load of 680 tons and a moment of 1193 foot tons in the direction of road and 968 foot tons in the perpendicular direction. Maximum pressure on the pier is 334 lb per sq. in. compressive and 36 lb per sq. in. tensile. As such, no vertical reinforcement is necessary. However, nominal reinforcement was provided for temperature and shrinkage effects.

(a) Haunches

The haunched portions between the outer walls of pier shaft and the cross walls are designed as columns carrying all the load from girders. With this assumption even though no reinforcement is required in the columns, 0.8 per cent reinforcement is provided in the section of the haunch.

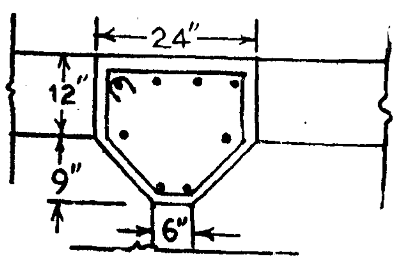


Fig. 1

(b) Piers walls

The piers being hollow, the outer walls are designed for horizontal water pressure which increases with the depth of pier. The walls span over 4 cross walls.

5.3. Superstructure

The section of the bridge with girders and the deck slab is treated as a homogeneous section subjected to dead load and live load bending moments. The same mix of concrete is used in girders and slabs. The deck slab, which is treated as the top

flange of the section, is provided with sufficient mild steel reinforcement assuming it to act as a R.C.C. slab continuous over girders and cantilevered out at the ends.

5.3.1. Distribution of load on the girders

A homogeneous section presupposes that any superload on the bridge will be borne by the entire section. This should result in identical bending stress $\left(\frac{M}{Z}\right)$ at any horizontal plane of the entire cross section. The identical stress can occur only when the reaction on each girder is equal (i.e., $R = \frac{W}{n}$, where n is the number of girders), irrespective of the position of load along the width of the deck. It is doubtful whether this effect could be achieved cent per cent. The share of load by each girder will vary, firstly, according to the distance of the load from the girder and, secondly, according to the nature of connection between the deck slab and the girder. The deck slab is rigidly connected to the girder but there is only a partial fixity between the two.

When I.R.C. Class 'A' trains move very close from the kerb line, the outer girder on that side shall take a greater share of the live load; whereas at other positions of the trains the inner girder may take the maximum share of load. But the results obtained by this analysis will have to be modified on account of the following considerations:

(i) Had there not been diaphragms rigidly fixed to the main girders at intervals, the unequal reactions might have created differential deflections in the girders. But the diaphragms try to equalise these differential deflections which, in turn, should influence the distribution of load on the girders. During the testing of Dimni bridge, equal deflections were recorded in all the four girders.

(ii) Secondly, the section of the bridge is comparatively much thinner than the section of a similar R.C.C. bridge of the same span. It has been noticed on Dimni and Didi bridges that even a small live load creates vibrations in the bridge. This indicates higher resilience in the structure.

It is rather difficult to calculate the maximum share of each girder exactly. For the purpose of design the effect of the live and dead load is assumed to be equal on all the girders but the live load moment is increased in order to account for such shifting of live load in the width of bridge that may create maximum reactions on the inner or outer girders. The ratio of the 'design live load moment' and the 'actual live load moment' is termed as the **Side shift factor**.

5.3.2. *Side shift factor*

For a particular clear roadway, the side shift factor is governed by the number of girders and their spacing. It is taken as the mean of the maximum reaction factors for inner and outer girders. The position of trains in each case would be obviously different. Two support conditions are assumed in the calculations :

(i) The deck slab resting on elastic supports of girders.

(ii) The deck slab acting as a continuous slab over firm supports of girders.

Maximum reaction factor for, say, an outer girder, is the mean value of reaction factor obtained in case (i) and case (ii). Fig. 2 gives the position of live loads that would create maximum reaction on the girder under consideration. The values are given in Table 3.

TABLE 3

Side shift factors

For Class 'A' Loading, 2 lanes

Condition of support	Reaction factors			
	Four girder system		Three girder system	
	Outer girder	Inner girder	Outer girder	Inner girder
Elastic supports ...	0.642	0.549	0.831	0.670
Firm supports ...	0.732	0.493	0.727	0.852
Mean reaction factors ...	0.688	0.521	0.779	0.761
Mean reaction on each girder ...	0.605		0.773	
Total effective load of 2 trains...	2.418		2.32	
Average side shift factor ...	1.209		1.16	

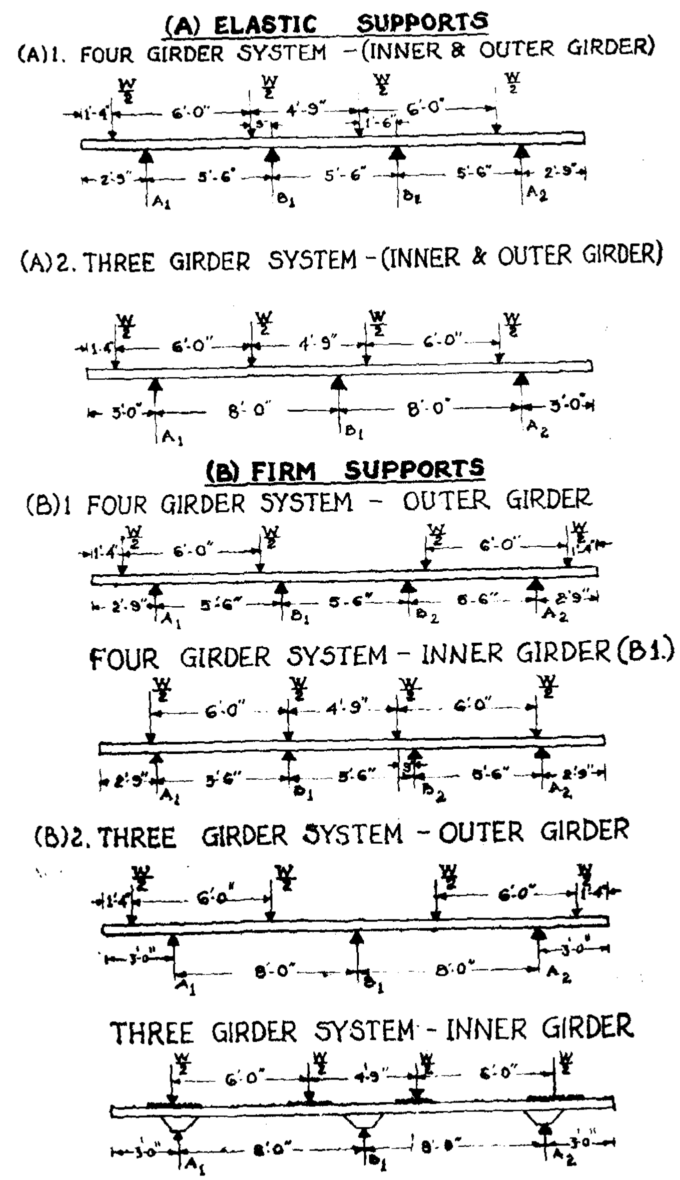


Fig. 2. **Position of Class "A" loads for maximum reaction**

5.3.3. Girder system

The analysis given in Table 3 reveals that the average side shift factor for four girder system is 1.209. It means that the distribution of live load on account of the arrangement of girders is such that homogeneous section of girders and slab is required to be designed for 20.9 per cent more bending moment than the actual bending moment of two trains. It can, therefore, be argued that if the number of girders in a fixed roadway is increased, the total load on each girder is not reduced in the same proportion. It follows logically that the lesser the number of girders, the smaller is the side shift factor. It is obvious that in one girder system the side shift factor is unity. The factor varies directly with the number of girders. From this view point one girder system is the most economical system of girder and slab bridges; but this cannot be adopted extensively on account of practical difficulties. Two girder system has been adopted economically at many places, e.g., Mahi bridge at Vasad in Bombay State.

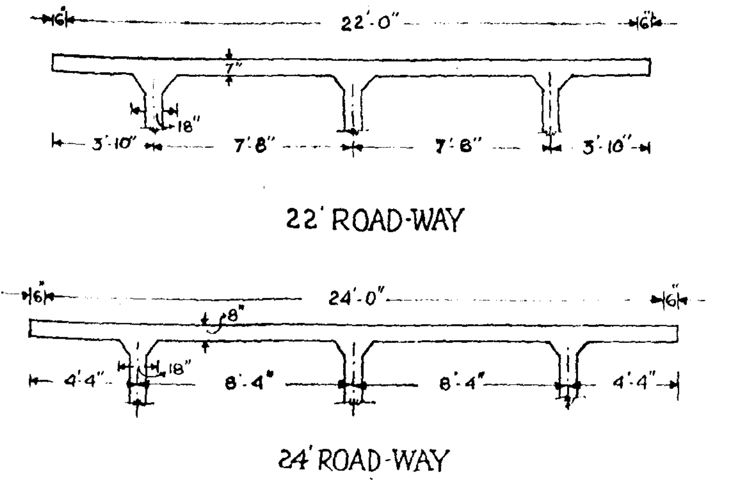
In the opinion of the Authors the three girder system gives a more rigid structure than the two girder system and is the best system of girder and slab bridges. The side shift factor for this system at Budhara bridge is 1.16 which is less than the same for the four girder system of Didi and Dimni bridges. Secondly in the three girder system the total dead load of girders and slab shall be less than the same for the four girder system which results in reduction of dead load bending moment also.

If economic design is carried out, the reduction in totaldesign bending movement of the three girder bridge in comparison with the four girder bridge would be between 5 to 10 per cent. Thus in the three girder bridge there is saving in the cost of concrete, steel and also centering as one girder is eliminated. Actual reduction in the overall rate of Budhara bridge in comparison with Didi may be seen in Table 7.

5.3.4. Economic spacing of girders

In a particular girder system for a fixed width of roadway, if the spacing of girders is varied, the reaction factors also vary accordingly. There is a particular spacing which may give the minimum reaction factors and hence most economical design for that system. This spacing should be such that the design of deck slab also is economical. The girders should be as close as possible because thereby the length of piers can be reduced which leads to economy in the cost of substructure. On investigating a number of alternative spacings in the three girder system for

22 ft and 24 ft wide roadways, the Authors are of the opinion that the spacings indicated in Fig. 3 give economical design.



RECOMMENDED SPACINGS OF THREE GIRDERS

Fig. 3

The reaction factors and side shift factors for these spacings are given in Table 4.

TABLE 4

Recommended three girder system

S. No.	Particulars	Reaction and side shift factors (Class 'A' loading, 2 lanes) 22 feet roadway spacing 7 ft 8 in. c/c Outer girder	22 feet roadway Inner girder	24 feet roadway spacing 8 ft 4 in. c/c Outer girder	24 feet roadway Inner girder
1.	Elastic supports ...	0.835	0.667	0.887	0.667
2.	Firm supports ...	0.768	0.817	0.834	0.811
3.	Mean reaction ...	0.801	0.742	0.860	0.739
4.	Side shift factor ...	1.158		1.200	

The recommended spacings and side shift factors can be adopted for R.C.C. as well as prestressed concrete girder and slab bridges of any span, subjected to I.R.C. Class 'A', 2 lane traffic. If the bridge is to be designed for Class 'A A' loading also, the same spacings will be all right. Maximum design live load moment may, however, change.

5.3.5. Stresses and ultimate strength

(i) **Stresses**—Properties of the central section of the girders, design moments and stresses at various stages for both types of girder systems are given in Fig. 4.

(ii) **First crack**—It is worth while to investigate the live load which will cause cracks in the bottom fiber. The first crack may appear when the total tensile stress is 500 lb per sq. in. In case of Dimni bridge, if extra load that would create a stress of 522 lb p.s.i. tensile at the bottom fibre is added, the section shall start cracking. Calculations indicate that this load is 44 per cent above the load of two Class A trains. The factor of safety against first crack is, therefore, 1.44 with respect to live load and 1.155 with respect to the total specified dead and live load.

(iii) **Ultimate strength**—Exact analysis for the ultimate strength of a prestressed concrete section under flexure is a complicated theoretical problem because both steel and concrete are generally stressed beyond their elastic range. For the purpose of practical design, empirical formulae based on the test results are available. Such formulae presuppose critical failure in the flexural. The behaviour of prestressed member after progressive strains causing tensile cracks in the region adjoining the cable is somewhat analogous to that of an R.C.C. member. Various formulae have been developed to predict the failure capacity of well balanced sections. According to the formulae specified in the "Criteria for Prestressed Concrete Bridges", the ultimate flexural strength in the case of the Dimni bridge is 2.14 times the design maximum bending moment. The factor of safety against ultimate strength is, therefore, 2.14.

5.3.6. Frictional losses and extensions

In post-tensioned system of prestressed concrete forces applied at the jack end are not fully transmitted in all the parts

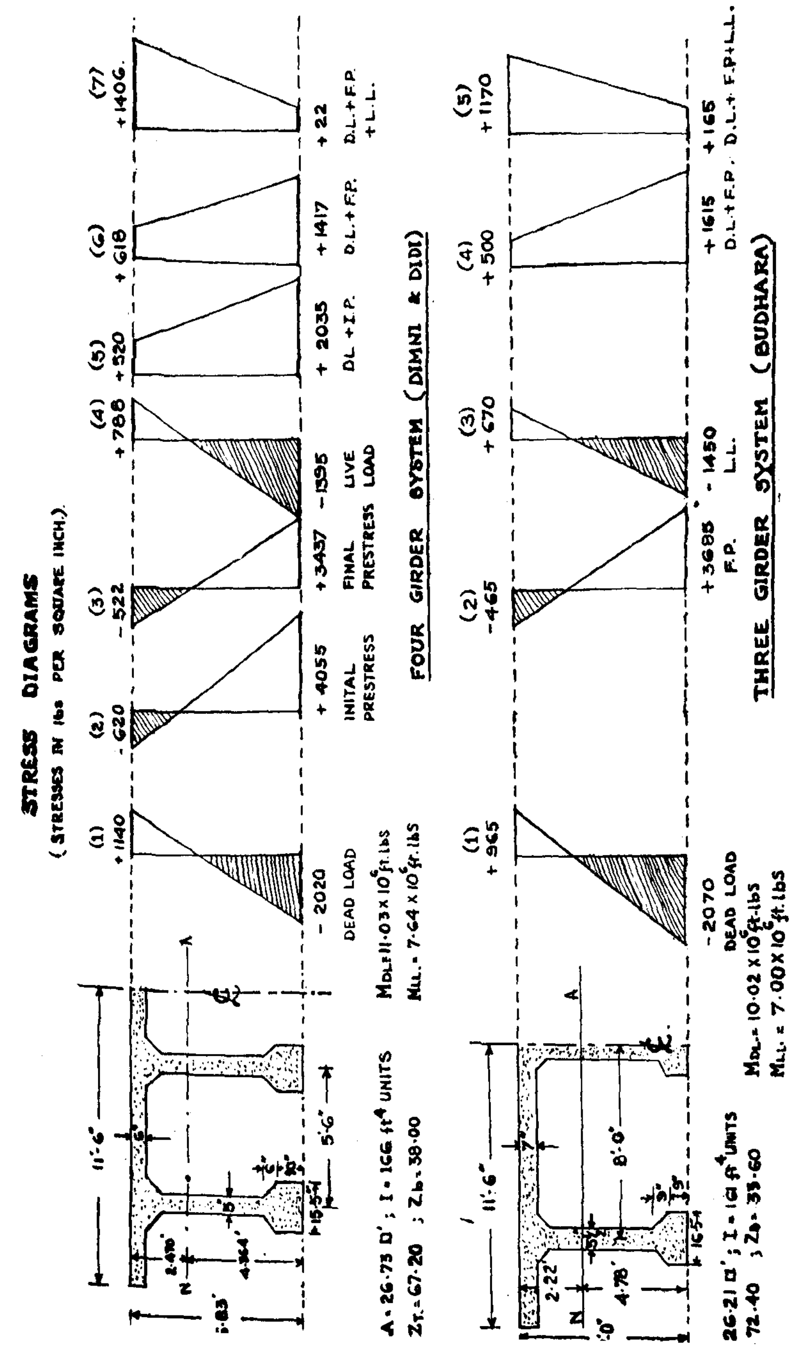


Fig. 4

of the cable on account of the friction developed (i) in the jack and anchorage, (ii) between cable surface and the body of sheathing material or concrete and (iii) on account of the designed profile of the cable. The friction is calculated by E.M. Cooley's procedure. Frictional losses are halved if the cables are stressed from both ends. Even then the total friction in some cables is as much as 20 per cent of the force applied. In such cases if a stress of 66 tons per square inch is to be developed at the centre of span, the pressure required to be applied at the jack end is $\frac{66}{0.8}=82.5$ tons per sq. inch. Table 5 gives actual recorded extension of one cable. It should be noted that from the recorded manometer readings at least 6 per cent to 8 per cent losses will have to be accounted for jack friction. Hence, actual stress on cable is about 93 per cent of the reading. For 420 kg/cm^2 the stress in the cable =81.00 tons per square inch.

TABLE 5

Extensions of cables

S. No.	Manometer reading in kg per sq. cm.		Elongation in inches Length of cable 130 ft 1½ in.		Total extension
	Right bank	Left bank	Right bank	Left bank	
1.	100	100	10/16	14/16	
2.	150	150	14/16	1—1/16	
3.	200	200	1—7/16	1—11/16	
4.	250	250	1—14/16	2—3/16	
5.	300	300	2—7/16	2—12/16	
6.	340	340	2—12/16	3—7/16	
7.	400	400	3—7/16	4—2/16	
8.	420	420	4—2/16	4—3/16	8—11/16

After locking and dewedging, locking pressure is constant at 480 kg per sq. cm. at both ends } 4—1/16 + 4—8/16 = 8—9/16

Note: Table 5 gives elongations of cable No. 5 in the inner girder of the first span at Didi bridge.

Elongation actually obtained ... 8—9/16 in.
Calculated elongation ... 8—1/2 in.

6. CONSTRUCTION

The unusual problems described below which came forth during the construction of these bridges were mainly two, "shifting and tilting of wells at Didi bridge" and "artesian fountain in the foundations of Budhara bridge." The sequence of casting the girders and stressing the cables also needs to be mentioned.

6.1. Shifting and Tilting of Wells at Didi Bridge

At Didi bridge one of the central wells was partially sunk before the monsoons of 1956. It was left in that state on account of floods. During the floods the well shifted bodily by 10½ inches and there was also some tilt. To remove the tilt, uneven loading was employed during the remaining sinking. When the well reached the depth of plugging, there was still a tilt of 3⅓ in. which being less than 1/60th of the depth was allowed. It was, however, not possible to remove the shift altogether. The defect was corrected by cantilevering out the well cap suitably towards the upstream side in order to orient the pier shaft correctly.

6.2. Artesian Spring in Well Foundation

(a) The spring

At Budhara bridge, while sinking wells for pier No. 3 near the right bank, an artesian spring having a constant discharge of 1.5 to 2 cusecs was encountered at R.L. 39.50, about 43 feet below the bed level. The spring had a hydraulic head of about 62 feet from the level where it was tapped. Dewatering the well by pumps for putting in 3 ft 6 in. bottom plug of colcrete was not possible in the powerful spring. The problem was rather formidable.

Such a phenomenon is not heard of in any of the bridge foundations in this region which is not known for artesian conditions. The causes of the spring, therefore, cannot be ascertained exactly. Artesian springs are generally encountered at greater depths. In this case the depth was only 43 feet. It would be a problem worth investigation by employing seismic methods to study the lower strata.

(b) Process of plugging the wells

The wells were sunk to the designed depth. Dredging in the wells was stopped when it became extremely difficult to excavate further. The main problem was how to carry out colcrete plug-

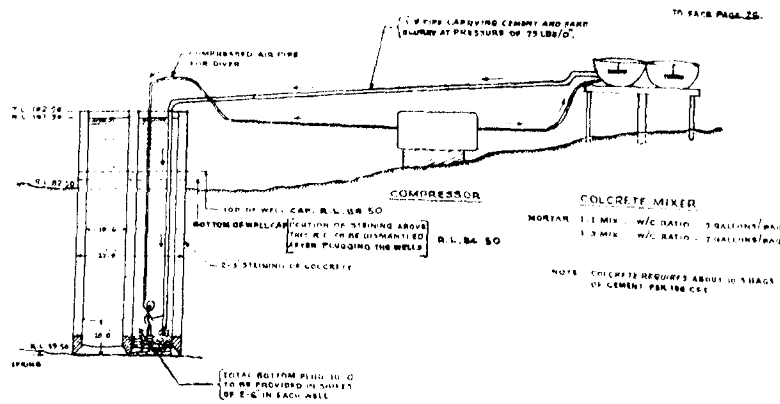


Fig. 5. **Plugging of wells in artesian spring at Budhara bridge, pier No. 3.**

ging under flowing water. This was effectively tackled by raising the well steining above the bed level upto R. L. 102.50 and making it fairly water tight. The spring water rose upto R. L. 101.50 and having lost its head and velocity remained steady at this level. When no more rise was observed for a few days, further work was commenced.

In one of the twin wells, a 2 ft 6 in. layer of boulders was packed at the bottom with the help of a diver. The diver was again sent inside with a hose pipe connected to a plant which supplied cement and sand slurry at pressure. At the bottom most section 1:1 cement sand slurry was injected at a pressure of 75 lb per sq. inch for a few inches and then 1:3 mix for the remaining depth. During colcreting it was observed that no particle of cement or sand came to the surface of clean spring-water. The same process was followed in other wells. Thus a total bottom plug of 10 ft was colcreted in 8 shifts (4 shifts for each well). Water in the wells was then pumped out and extra steining was dismantled. Fig. 5 illustrates the process of colcreting under water.

In other wells where such a spring was not tapped, only 3 ft 6 in. bottom plug was provided. In this case, 10 ft plug was felt necessary in order to withstand the upward thrust of the spring. This was purely a practical decision.

(c) Testing of wells

On account of the existence of strong spring below, it was feared that the spring might cause some trouble in future

Hence, before proceeding with further work, it was decided to test the effect of design dead and live loads on the well. The wells were loaded by means of sand bags weighing 668 tons. They were equipped with an arrangement to measure sinking to the accuracy of 1/24 in. After eight days of loading, no settlement was recorded. The work was, therefore, continued further with confidence.

The same spring was met with below pier No. 2 and it was dealt with in a similar way.

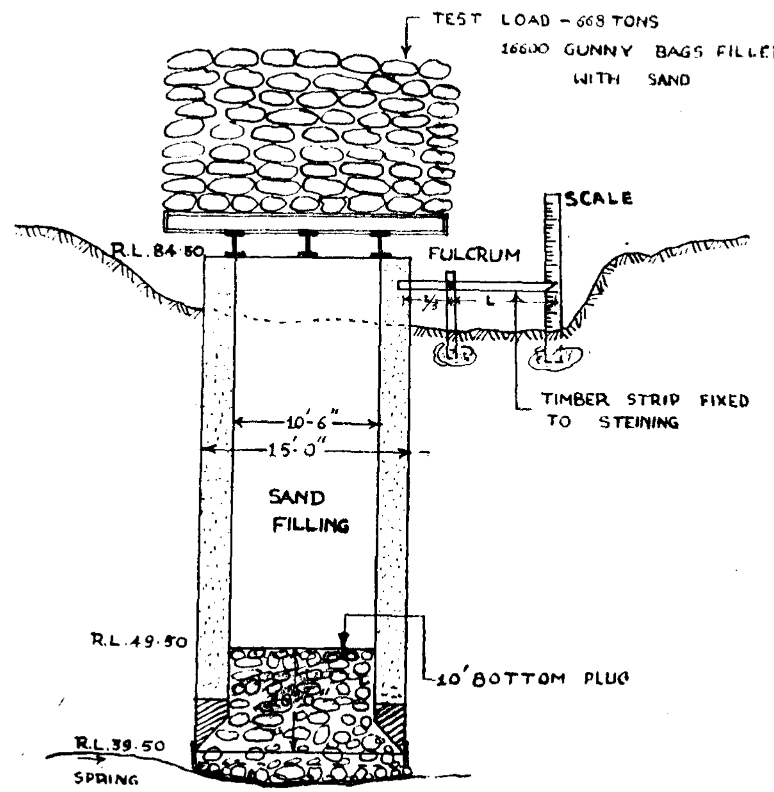


Fig. 6. **Budhara bridge, testing of wells at pier No. 3**

6.3 The sequence of concreting and stressing the cables

The procedure of concreting and the sequence of stressing cables in case of one of the bridges (Budhara) are as follows:

Casting of main girders of one span (127 ft centre to centre piers) required about 12 days. Seven days after this, the work of arranging reinforcement of deck slab was taken up which required about a week. Two lower cables in each girder were then stressed and the deck slab was concreted in the next week. Another 46 cables were stressed 7 days after the last batch of concrete was poured in the deck slab, at the rate of 8 cables per day. The bottom centering of girders and deck slab was then removed. The remaining 16 cables which project out through the deck slab were stressed after laying wearing coat and fixing the railings. The sequence of stressing was such that at no stage immediate temporary stress in the concrete was more than 2100 lb per square inch.

In case of Didi and Dimni bridges, out of the 74 cables, only 4 cables were left to be stressed after putting wearing coat and railings.

7. TESTING OF DIMNI BRIDGE

The construction of Dimni bridge was completed before the monsoon of 1958. There were no constructional difficulties as such. The extensions of cables actually obtained tallied with the theoretical values. Similarly, compressive strengths of 6 in. × 6 in. × 6 in. concrete cubes, taken from time to time during construction, were in no case less than the specified strength. Yet, this being the first prestressed concrete work constructed by Madhya Pradesh P.W.D., it was decided to test the bridge for full live load. The test was carried out by the simple field method recommended by Shri S. V. Natu *. That Paper recommends that the bridge should be tested for three loads.

1. Full live load with impact.
2. For load causing permissible tensile stress at the bottom.
3. For load causing first crack.

In case of Bhima bridge at Takli, the extensions in some cables were reported to be less than the theoretical extensions and some cables were broken while prestressing. On acount of these defects, perhaps, Bhima bridge was tested for the above loads. Since no such difficulties arose at Dimni, testing for full live load with impact was considered adequate.

Table 6 gives the result of the test which is illustrated by photographs. A load of 265,000 lb spread over an area of 22 × 20 ft placed at the centre of the span, would cause design

* Paper No. 189—"Load Testing of the Prestressed Concrete Bridge across Bhima River at Takli on Sholapur—Bijapur Road", Vol. XXI-Part 1 of the I.R.C Journal, February 1957.

TABLE 6

Load test-deflections

Particulars	Beam 1	Beam 2	Beam 3	Beam 4
1. DEFLECTIONS WHILE INCREASING THE LOAD FROM 0 TO FULL LOAD (17-7-58)				
25 per cent load	0.75	0.75	0.75	0.75
50 per cent load	1.60	1.60	1.60	1.60
75 per cent load	2.50	2.50	2.50	2.50
100 per cent load	3.50	3.50	3.50	3.50
2. DEFLECTION 24 HOURS AFTER LOADING	4.00	4.00	4.00	4.00
3. UNLOADING AFTER 36 HOURS ON 19-7-58				
At the beginning	4.30	4.30	4.30	4.30
75 per cent load	3.50	3.50	3.50	3.50
50 per cent load	2.90	2.90	2.90	2.90
25 per cent load	2.00	2.00	2.00	2.00
0 per cent load	0.40	0.40	0.40	0.40
4. DEFLECTION 8 HOURS AFTER UNLOADING	0.30	0.30	0.30	0.30

Actual deflection $\frac{4.30}{5} = 0.86$ in.

Theoretical deflection 1.1 in.

live load bending moment of 7.64×10^6 ft lb at the centre. This loading was put on 17th July 1958 and readings were taken at an interval of 25 per cent load. After maintaining the full load for 36 hours, unloading was commenced on 19th July 1958. The deflections at various loadings in all the girders were found identical. The maximum deflection was 0.86 in. It was less than the theoretical value which is 1.1 in. But the theoretical value is calculated by assuming a uniform section throughout the girder. Actually, the section increases gradually towards the end of span. Considering this fact, the difference between actual deflection and calculated value is not appreciable.

8. STATISTICS

8.1. Comparative Study

Tenders for Dimni, Budhara and Didi bridges were invited on the basis of R. C. C. bridge designs prepared by the Public

Works Department. For Dimni bridge, the Department proposed 6 spans of R. C. C. T beam and slab spaced at 66 feet centre to centre and supported on coarsed rubble masonry piers (1 : 6 cement mortar), which were to rest on twin well foundations of 1:3:6 cement concrete. This alternative would have required :

Cement	...	...	864 tons
Mild steel	...	...	150 tons

whereas the approved design of Messrs Gammons required the following quantities apporoximately :

Cement	520 tons
Mild steel	110 tons
H.T. steel	16.5 tons

It should be noted that the approved design provided R.C.C. substructure which consumed considerable quantities of steel and cement in comparison with the coarsed rubble masonry substructure of the alternative. Even then, there is appreciable saving in steel and cement. This is, firstly, on account of reduction in the number of piers and wells and, secondly, on account of the adoption of prestressed concrete superstructure.

8.2. Economy on account of three girder system

Table 7 gives approximate requirements of cement and steel for the 'entire bridge' and also for the superstructure only of the Dimni, Budhara and Didi bridges. Approximate total cost of bridge proper and per running foot, square foot and cubic foot costs in each case are also included in the Table. It is quite clear that probable consumption of cement and H.T. steel and per running foot cost of Budhara bridge where three girder system is adopted is less than that for four girder bridge at Didi. The statistical figures thus support the discussion in para 5.3.

8.3. General statistics

Average rate of these bridges works out to Rs 1,685 per running foot. This is based on the rates of cement and steel prevailing in 1956. Since the present rates of these materials are about 30 per cent higher than the rates on which the tenders were based, probable present rate of a similar bridge would be Rs 1,800 per running foot. Percentage cost of cement and steel shall be approximately as given in Table 8. Table 8 also indicates per running foot requirements of these materials.

TABLE 7

Statistical data of bridges

S. No.	Bridge with length	Average height of bridge	Area of roadway on bridge	Cement (tons) Total	Cement (tons) Per 100 sq. ft.	Mild steel (tons) Total	Mild steel (tons) per 100 sq. ft.	H. T. steel (tons) Total	H. T. steel (tons) per 100 sq. ft.	Cost of bridge proper (in lakhs)	Cost per rft. Rs	Cost per sq. ft. of roadway Rs	Cost per sq. ft. per ft. height (per cu. ft.) Rs
1.	Dimni 392 ft 2 in.	83 ft	8628 sq. ft.	520 (210)	6.00	110 (42)	1.29	16.5 (16.5)	0.191	6.30	1610	73.2	0.88
2.	Budhara 504 ft 0 in.	85 ft	11088 sq. ft.	610 (231)	5.5	150 (54)	1.35	19.5 (19.5)	0.176	8.50	1685	76.5	0.90
3.	Didi 524 ft 2 in.	85 ft	11532 sq. ft.	700 (280)	6.1	150 (56)	1.30	22.0 (22.0)	0.191	9.13	1745	79.4	0.94
4.	Totals & averages 1420 ft. 4 in.	—	31248	1830 (721)	5.85	410 (162)	1.31	58.00 (58)	0.186	23.93	1685	76.5	0.91
5.	Steel & cement per rft. (average)	—	—	1.29 tons (0.507)		0.29 tons (0.107)		0.0408 tons (0.0408)					

Note :—(1) Figures given in the Table are not exact figures but approximate quantities of materials and costs.
(2) Figures in the brackets indicate the quantities of materials in the superstructure only.

TABLE 8

Consumption of cement and steel

(In entire bridge)

S. No.	Material	Quantity per rft. of bridge tons	Per cent cost on the 1956 rate of Rs 1685 per rft.	Per cent cost on the probable present rate of Rs 1800 per rft.
1.	Cement	1.29	7.6	9.3
2.	Mild steel	0.29	12.0	14.7
3.	H. T. steel	0.041	3.0	3.7
4.	Remaining materials and labour etc.	—	77.5	72.3

9. CONCLUSIONS

On studying various aspects of the design and construction of bridges across Kunwary river, principal conclusions of the Authors are as follows :

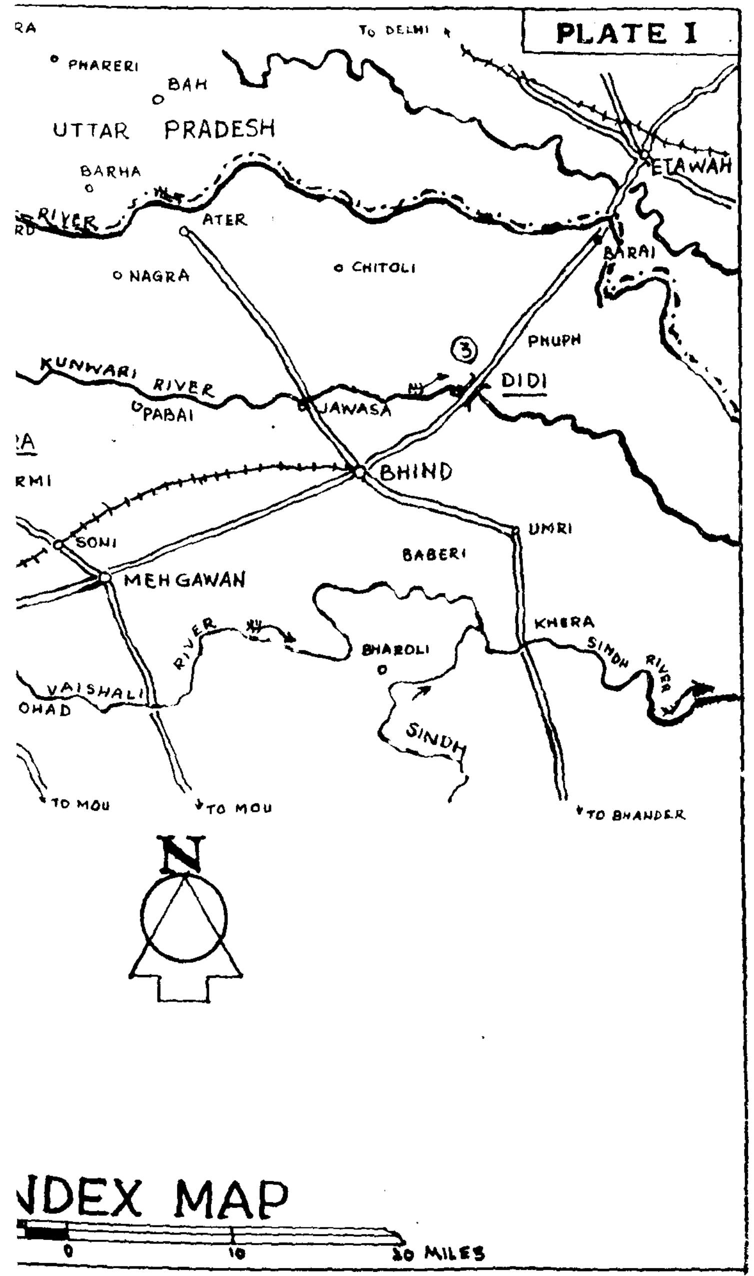
1. The design flood discharge at a bridge site should be related to the maximum intensity of rainfall, time of concentration and other hydraulic characteristics of the catchment. Empirical formulae alone should not be relied upon.

2. Three girder bridge is economical than the four girder bridge for 2 lane traffic. If the girders are suitably spaced, maximum economy in the superstructure as well as in the substructure is possible. In the opinion of Authors, 8 ft 4 in. spacing for 24 feet roadway and 7 ft 8 in. spacing for 22 ft roadway can be treated as ideal spacing.

ACKNOWLEDGMENT

The Authors are grateful to Shri P. V. Divatia, Chief Engineer, Madhya Pradesh, Public Works Department (B. & R.) Branch, for his encouragement to write this Paper and the permission to publish the same. They acknowledge with thanks the assistance rendered by Messrs Gammons India Private Ltd., who are responsible for the design and construction of these bridges.

Acknowledgments are also due to Shri A. M. Kanhere, Executive Engineer, Bhind and Shri D. R. Pathak, Executive Engineer, Morena in charge of the construction works for supplying necessary information.



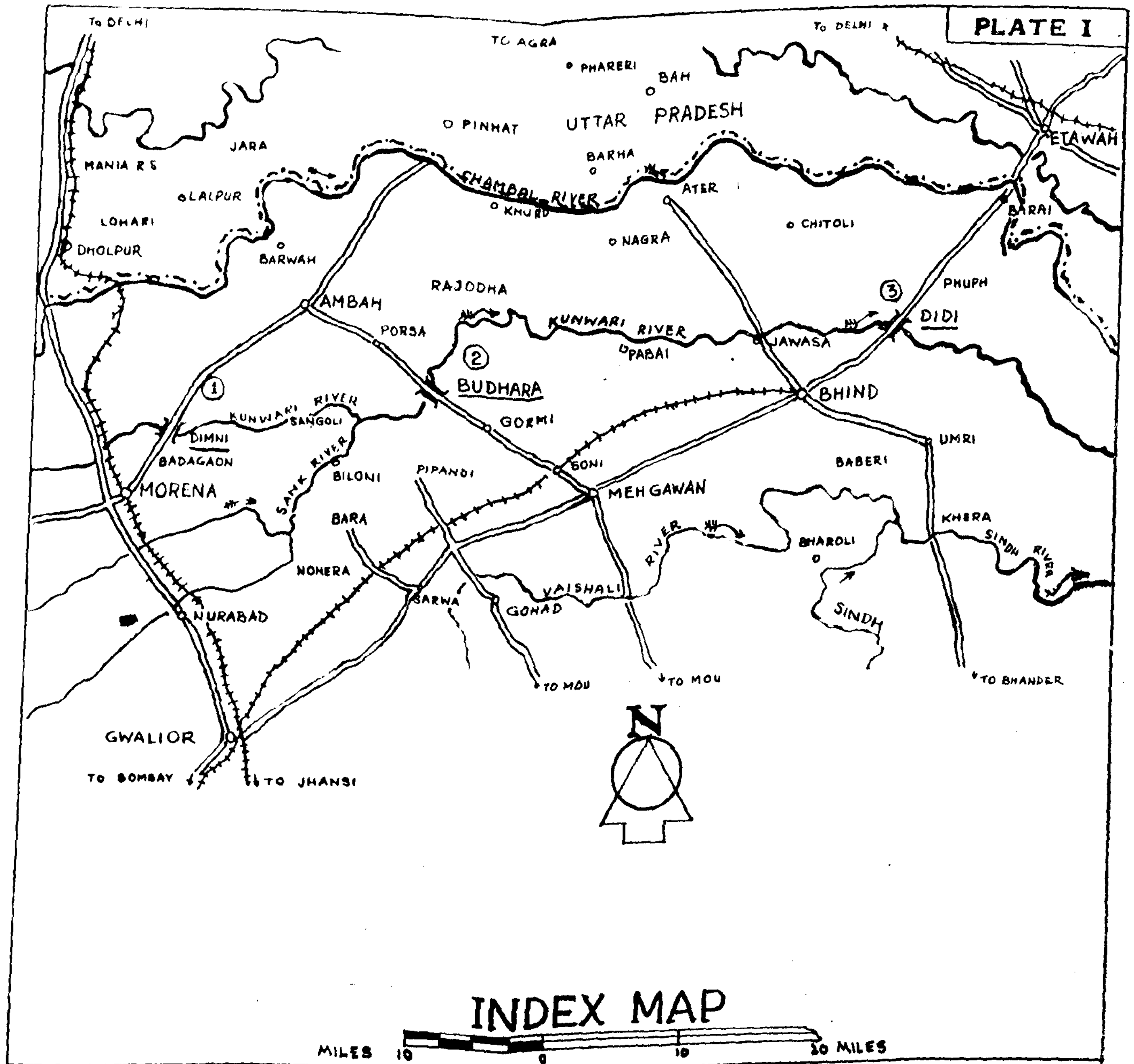
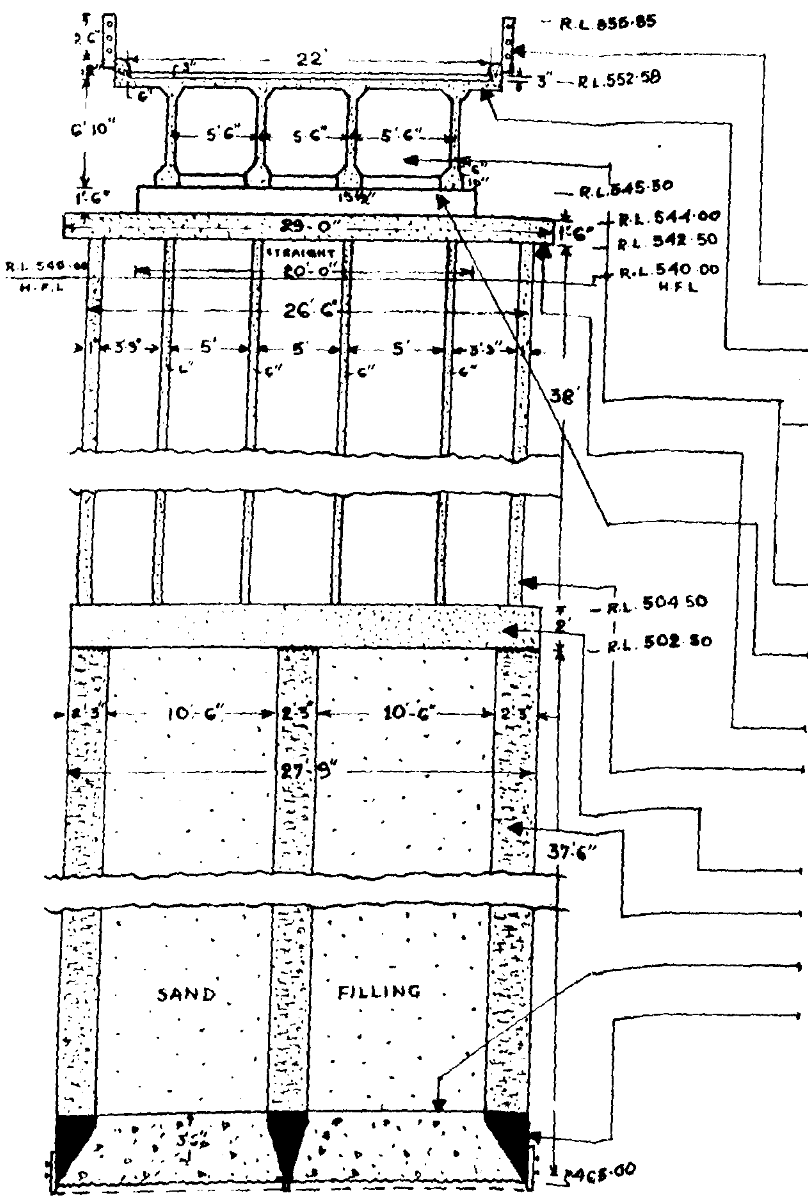


PLATE I
TO DELHI
TO AGRA
PHARERI
BAH
PINHAT
UTTAR PRADESH
ETAWAH
JARA
MANIA R S
LALPUR
BARHA
CHAMBAL RIVER
ATER
KHURD
LOHARI
CHITOLI
BARAI
DHOLPUR
NAGRA
BARWAH
RAJODHA
PHUPH
AMBAH
KUNWARI RIVER
DIDI
PORSA
PABAI
JAWASA
BUDHARA
BHIND
KUNWARI RIVER
SANGOLI
DIMNI
GORMI
UMRI
BADAGAON
SONI
BABERI
MORENA
BILONI
PIPANDI
MEHGAWAN
SANK RIVER
BARA
KHERA
SINDH RIVER
BHAROLI
NONERA
VAISHALI RIVER
BARWA
GOHAD
SINDH
NURABAD
TO MOU
TO MOU
TO BHANDER
GWALIOR
TO BOMBAY
TO JHANSI
N
INDEX MAP
MILES 10 0 10 20 MILES

PLATE II



CROSS SECTION

STRUCTURAL DETAILS

PIPE RAILING WITH 3 G.I. PIPES PASSING THROUGH R.C.C. POSTS SPACED @ 6' C/C

6" THICK DECK SLAB OF SPECIAL GRADE R.C.C. 1:1½:3 MIX (THE SLAB IS HOMOGENIOUS WITH THE PRESTRESSED GIRDERS)

PRESTRESSED GIRDERS, 4 NOS. OF SPECIAL GRADE CONCRETE 1: 1½: 3 MIX WITH 74 FREYSSINET CABLES (EACH CABLE CONSISTS OF 12-0.2" Φ H.T. WIRES)

WEB THICKNESS OF GIRDER BULGING FROM 5" AT CENTRE OF SPAN TO 15½" AT THE END SUPPORTS

R.C.C. DIPHRAGMS 6"x 5'6", 9 DIPHARAGMS PER SPANS (1 : 2 : 4 H.G.)

R.C.C. ROLLER & ROCKER BEARING 1'6" x 0.9" WITH LEAD SHEETS (MIX 1:1:2)

R.C.C. PIER CAP 29x5'x1'-6" WITH SEMICIRCULAR ENDS (1:2:4. H.G.)

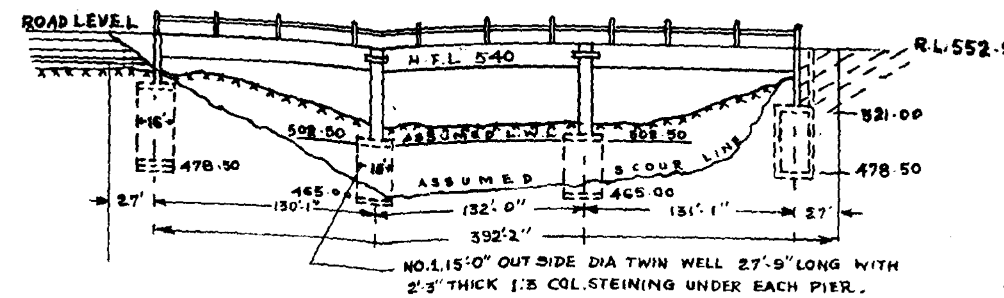
R.C.C. SHELL TYPES CELLULAR, PIER 26'-6" x 6'-6" WITH 6" R.C.C. DIPHRAGMS @ 5'-6" C/C BELOW THE MAIN GIRDERS, OUTER WALL OF SHELL 12" (MIX 1:2:4 H.G.)

R.C.C. WELL CAP 27'9"x15'0"x2'-0" MIX H.G. 1:2:4

WELL STEINING, COLCRETE 1:5. 2'-5" THICK.

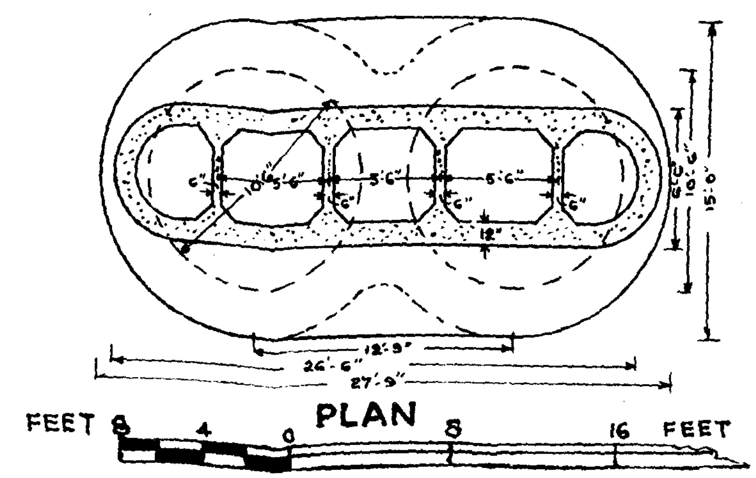
BOTTOM PLUG COLCRETE 1:5, 3'-6" THICK

R.C.C. KERB WITH STEEL CUTTING EDGE MIX 1:2:4 H.G.



ELEVATION

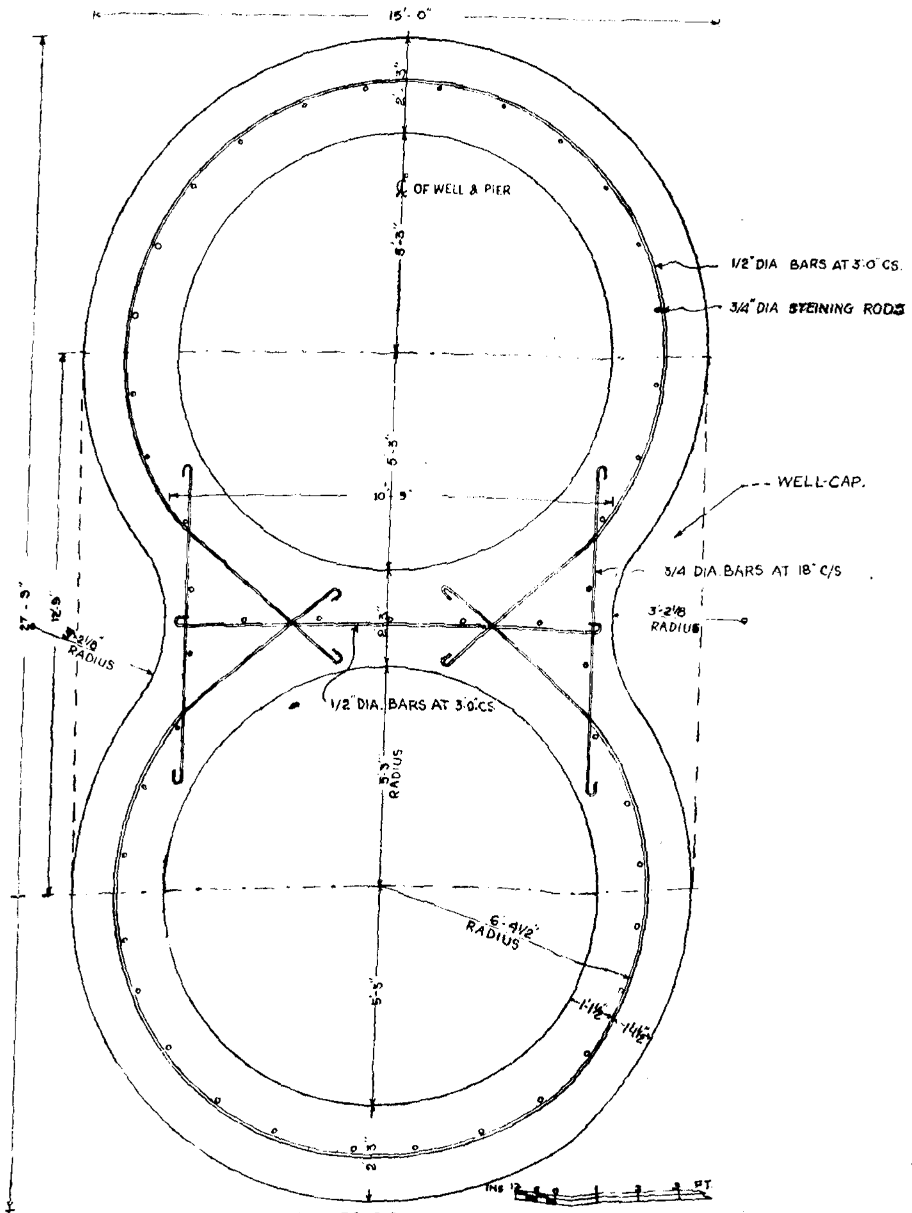
SCALE 1"=100'



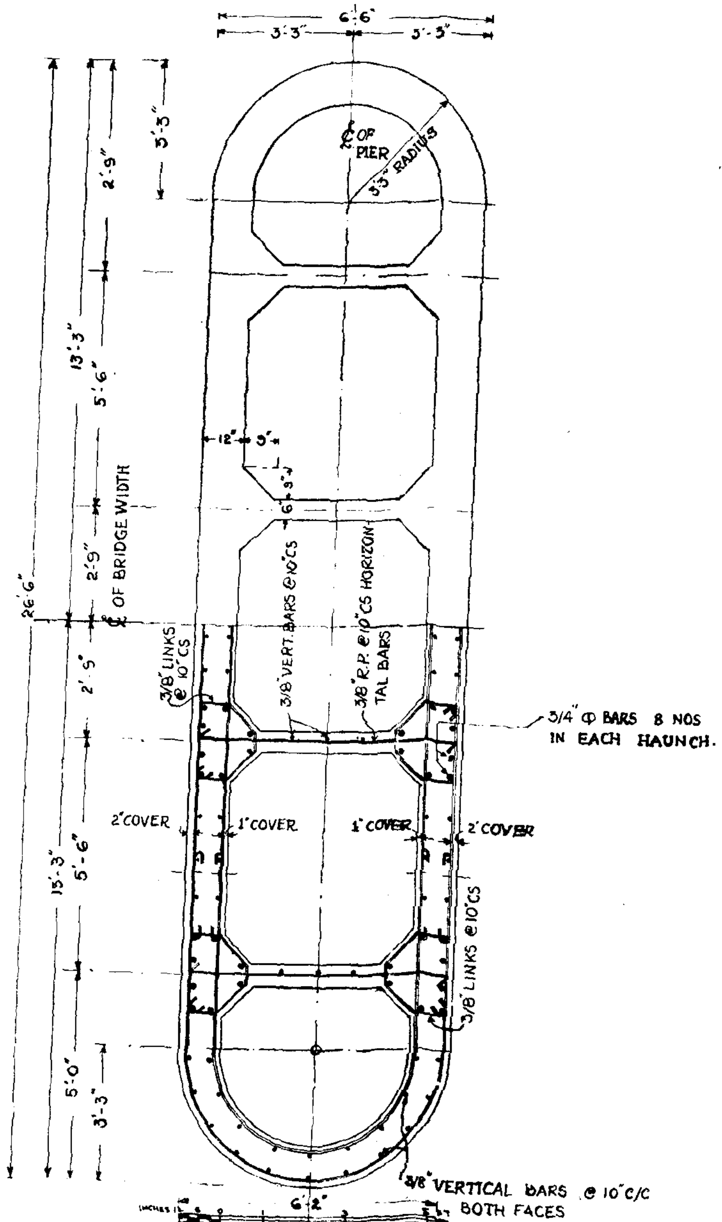
PLAN

DETAILS OF DIMNI BRIDGE.

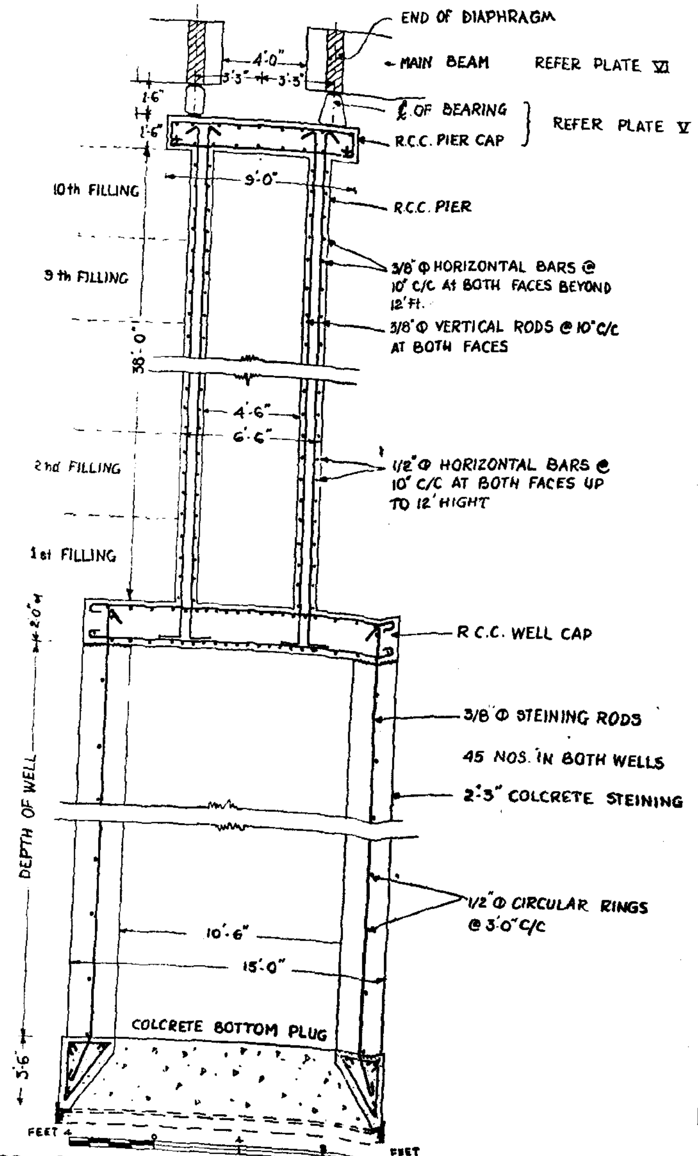
PLATE III



DETAILED PLAN OF WELLS



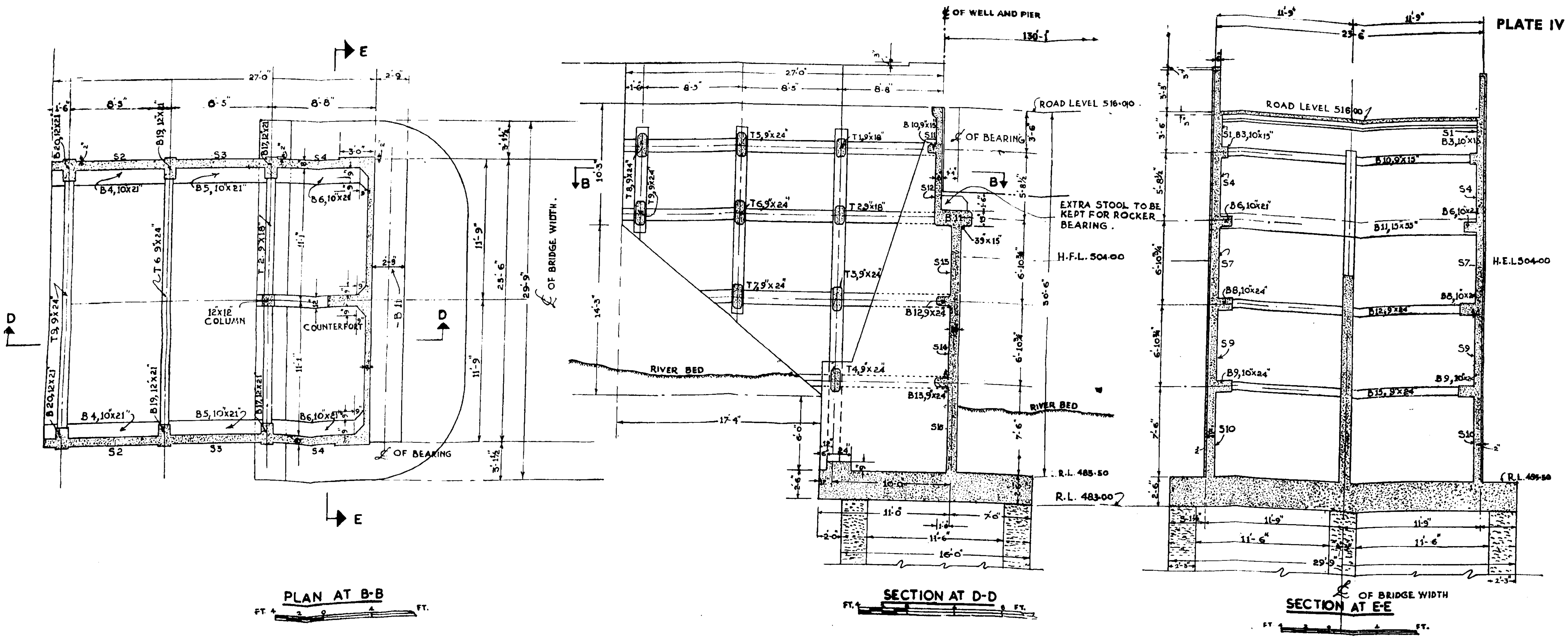
PLAN OF PIER SHAFT



CROSS SECTION THROUGH WELL & PIER.

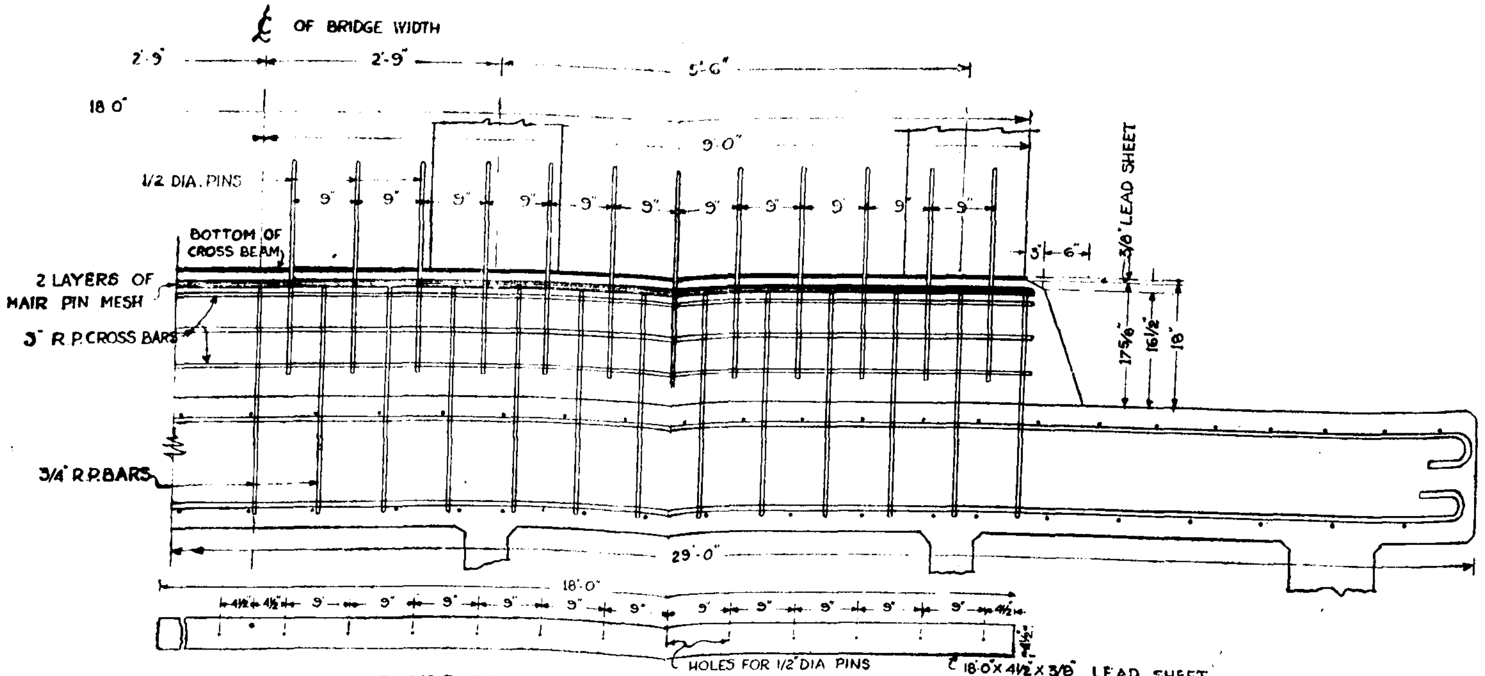
DETAILS OF SUBSTRUCTURE, DIMNI BRIDGE

PLATE IV

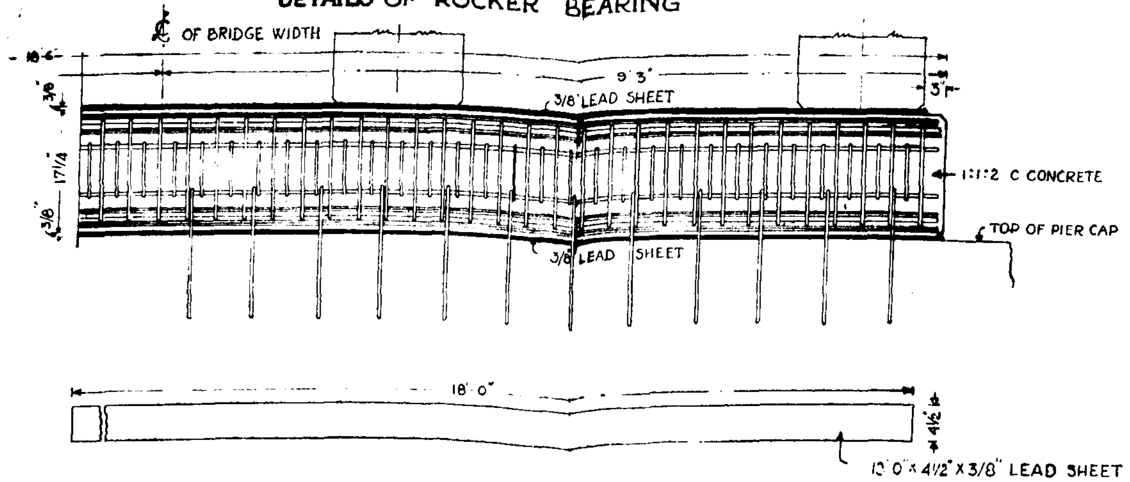


DETAILS OF END ABUTMENT (AT DIDI)

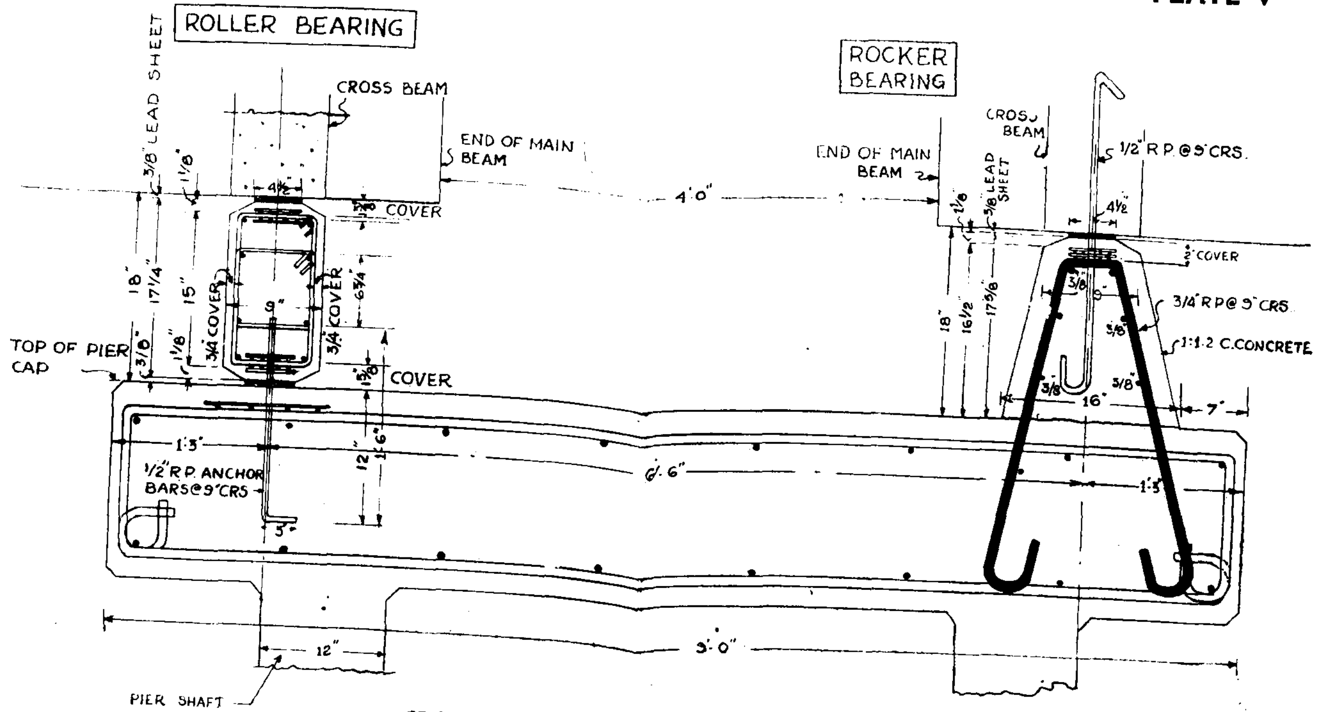
PLATE V



DETAILS OF ROCKER BEARING



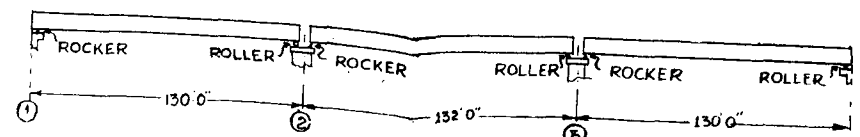
DETAILS OF ROLLER BEARING



CROSS SECTION THROUGH PIER-CAP

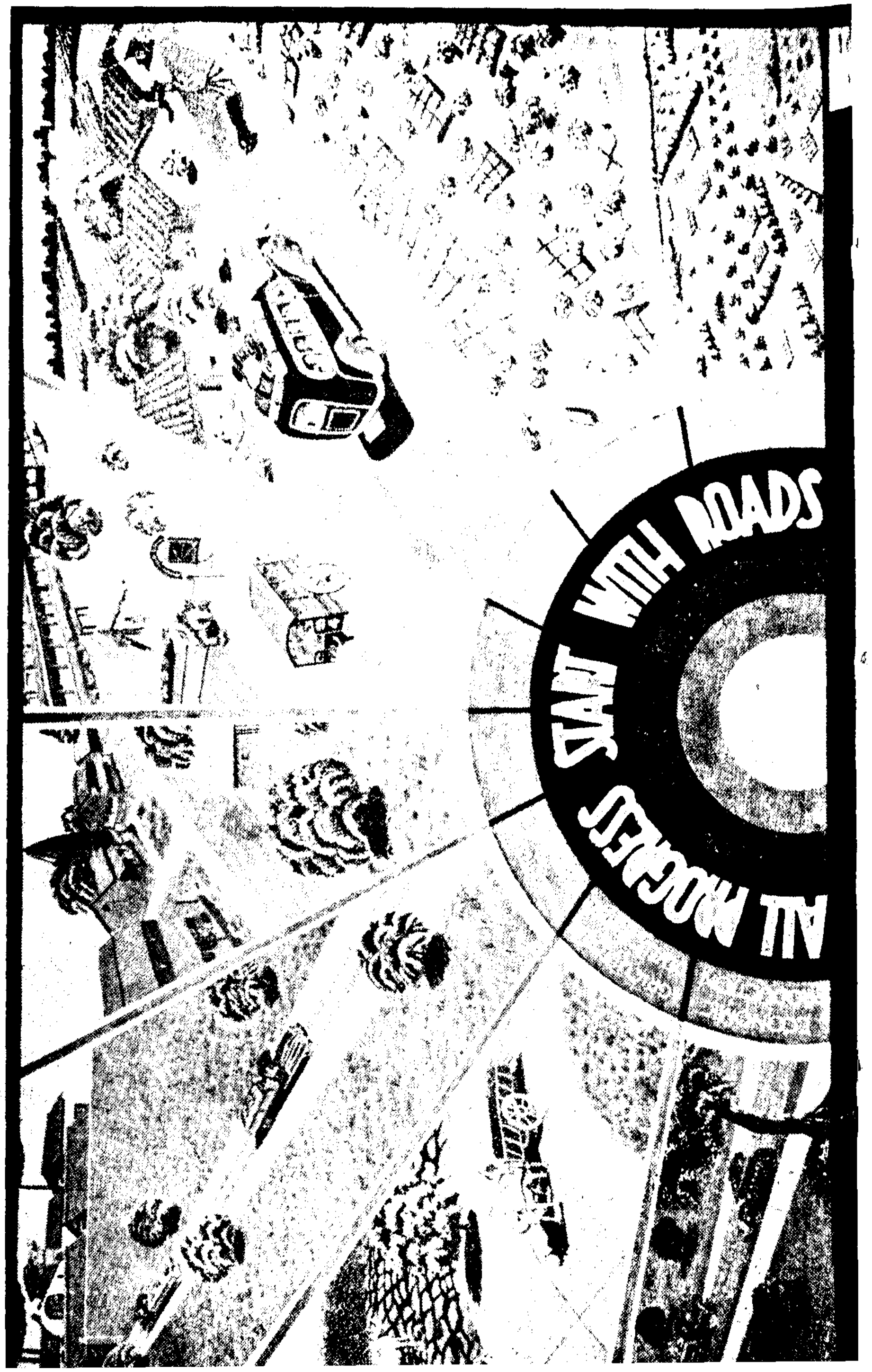
NOTE:—

1. DETAILS OF PIER-SHAFT ... REFER PLATE III.
2. DETAILS OF MAIN GIRDERS ... REFER PLATE VI.



KEY-PLAN FOR ROCKERS & ROLLERS FOR DIMNI BRIDGE

R. C. C. ROLLER & ROCKER BEARINGS, DIMNI BRIDGE



Paper No. 218

"CRYSTAL PATTERN FORMULA FOR INTEGRATED HIGHWAY PLANNING IN INDIA"

By

K. C. MITAL*

CONTENTS

SYNOPSIS

The Paper presents an approach to the problem of integrated highway planning, based on a pattern of roads designated as "Crystal Pattern", with special reference to Uttar Pradesh.

The fundamental requirements of highway planning in respect of accessibility to a road and road milage on area basis are considered and the application of these principles in the evolution of the Crystal Pattern Formula of road planning is described.

The merits of this pattern are discussed in detail and the targets of different classes of roads obtained by its application are given.

The preparation of a Master Plan of roads for any area by the application of the method described in the Paper is also explained.

1. INTRODUCTION

1.1. Highways are the life-lines of a country and provide facilities for the free and economic flow of men and materials

*Executive Enginner Planning, Public Works Department, Uttar Pradesh.

from place to place. Apart from the transport of commodities, a good highway system also helps in the cultural, social and ethical development that follows. Roads are also required for the development of industries in the countryside or to bring an area under cultivation or habitation. It is rightly said that "*All progress starts with roads. They lead and civilisation follows.*"

1.2. It is true that roads have existed ever since the dawn of civilisation. The movement of human beings in different directions created tracks which later developed into roads of some form or other in different ages. In India, as in other countries, the modes of transport in riverain areas were primarily boats, and pack-animals were used in the interior. The bullock cart can no longer cope with the increasing demand of the present age nor the requirements of the future in which motor transport will play an increasingly important part. An efficient system of motor transport requires a network of good roads.

1.3. The economic structure of the country will be severely disorganized if the overall transport capacity of the country—be it by Air, Rail, Road or Water Transport—is not developed quickly. In the year 1950-51, the total transport capacity of the country was:

Railways	19,225	passenger coaches
	199,094	goods wagons
Road Transport	34,411	buses
	85,799	goods and other vehicles
	9,600,000	bullock carts
Water Transport	217,202	India-owned tonnage of coastal shipping.

This needs further development to meet the increasing future demands of the country.

1.4. In the sphere of development of our highways, a relative study of the statistics, as compared to other advanced countries, given in Appendix I, reveals that India stands at the lowest rung of the ladder. The country needs to develop its communication system at least fourfold to cope with the growing transport requirements.

1.5. In 1943, the Nagpur Plan was drawn up on the basis of Grid and Star Formula and the targets were then fixed to match the needs of the next 20 years or so, but the develo pmen

in the country since Independence has been more rapid than anticipated and has necessitated the evolution of a more scientific and economical approach to road development. The Crystal Pattern Formula has, therefore, been evolved to obtain the outlines for drawing up detailed road development schemes for any State, or India as a whole, based on a scientific approach to the problem.

2. PLANNING AS APPLIED TO ROADS

2.1. Roads occupy an important place in the development of a country; their selection, construction or improvement should not be haphazard to become wasteful at a later date but should be planned rationally in a scientific and economic manner to serve the very core of the nation—the rural areas which are the bulwark of the country. To accomplish this task it is obligatory that the economic conditions as they exist today and as they are expected at the end of the next 20 or 30 years with the present growth of population and production, are intelligently investigated and studied together with the geographical and topographical conditions in the country.

2.2. Economic and Scientific Planning

2.2.1. For a study of the factors that assist in the scientific and economic planning of a highway system, the first stage is to collect the relevant statistics relating to the basic factors which govern the fixation of targets of road milage in a particular area for different types of roads, their alignments to suit the economic needs of transport, and the intensities of traffic to determine the thickness of road crust, road width, type of surface, etc., to suit the traffic requirements.

2.2.2. The first part is accomplished by a Factual Survey of the areas known as "Fact Finding Survey". Intelligent interpretation of the results of such surveys determines other guiding factors. These have already been discussed briefly by Shri P. N. Srivastava, Superintending Engineer, P.W.D., Uttar Pradesh, in his Paper No. 186 published in the Indian Roads Congress Journal, Vol. XX-3, December 1955. The information is collected and compiled separately under different heads given in Appendix II and is plotted on different maps, as given in Appendix III, for each district as a unit and is then studied in relation to the adjoining districts.

2.2.3. The factual information plotted on the maps will indicate at a glance:

(a) the existing traffic, its direction of flow, intensity and future trends of development;

(b) the potentialities of the areas in different sectors and their need for development of the communication system ;

(c) present importance of different sectors in relation to each other and their scope for future development.

2.2.4. A study of these maps will further indicate that:

(a) there is a tendency of direct links between towns and villages having a population above 2,000 in plains, above 1000 in semi-hilly areas and above 500 in hilly areas, unless they are obstructed by some unsurmountable natural impediments, and also for each village or town to link itself to its nucleus ;

(b) the villages below 500 population are scattered in productive areas and they cluster round bigger towns in plains as their satellites. In hilly regions, the spread over of small villages is observed more in valleys wherever small areas are available for cultivation ;

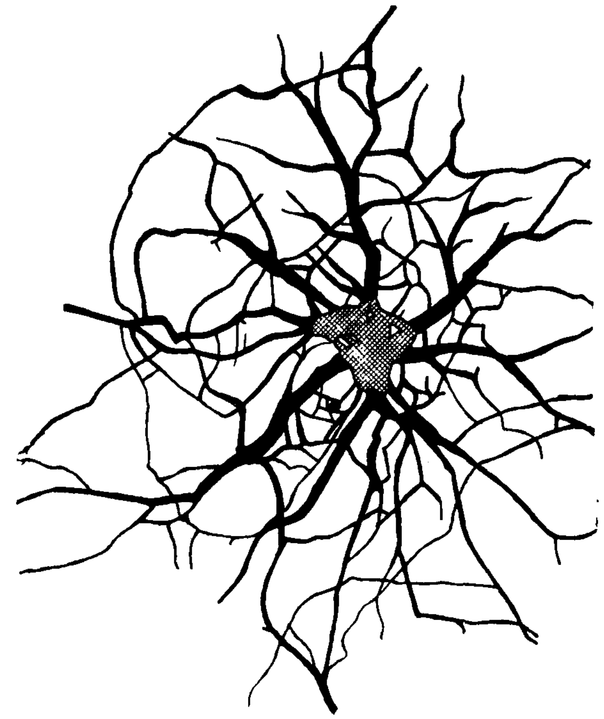
(c) the number of roads radiating from each village or town depends upon the potentiality and population of the place;

(d) the dispersal of traffic in the countryside is like the roots of a tree spread over the entire area and converging to a nucleus point, which is symbolic of a Grid and Star pattern, and which indicates how the highway plan should be developed so that it will best suit the needs of the country ;

(e) every big village or town has its own zone of influence within which traffic is concentrated towards a nucleus both for inward and outward traffic, and it is from this nucleus that traffic proceeds likewise to other bigger towns ;

(f) traffic intensity is in proportion to the population and produce in the area and develops wherever production or any industry or commercial centre is developed ;

(g) natural obstructions like rivers have impeded development of certain areas which can be developed by bridging such gaps on the river crossings. Wherever traffic has not developed in the absence of proper means of communication,



TYPICAL TRAFFIC VOLUME DISPERSAL DIAGRAM.

such areas have become the abode of dacoits and are a source of menace to the social life of the country ;

(h) there are places in hilly or semi-hilly areas which are capable of development as health resorts or which are rich in raw materials but have not been explored for want of proper means of approach.

3. FUNDAMENTALS OF HIGHWAY PLANNING

3.1. The bullock cart is an ancient heritage for transport in India. The substitute for an animal drawn vehicle is a power driven vehicle which has still to develop fully in this country. But there is no denying the fact that even the bullock cart today needs better roads to raise its economic value and it has to be co-ordinated with motor vehicles. Railways should not be afraid of the development of roads as they have their own functions to perform and both the systems can develop side by side and be more efficacious by mutual co-ordination. Construction of roads will foster the development of rail transport by making railways approachable to the interior where they themselves cannot reach directly.

3.2. It is, therefore, very necessary that a very clear assesment of the future trends of development should be made at this stage and basic fundamentals of highway planning studied carefully to draw up a plan of development of the future highway system in the country with proper co-ordination of all other means of transport.

3.3. Accessibility milage relation in the layout of highway system

3.3.1. The design of a road system is dependent upon the accessibility intended to be achieved, be it to any road or to any place. In the latter case, the target of road milage may become infinite and it is for this reason that the accessibility to a road should be considered.

3.3.2. The approach to all places should be as direct as possible within the framework of the road system so that the length is minimum and thus economical to construct and maintain.

3.3.3. The length of the road system is dependent on the form of its layout and vice versa. It is expedient that the road pattern should be such that every place is directly accessible in all directions with minimum detour.

3.4. Principles of growth

3.4.1. Road planners must take into consideration the principles of growth inherent in highway traffic. It will be uneconomical to design for traffic expected a century ahead, but a gradual upgrading with a clear conception of the future requirements will be economical so that road improvement is in balance with traffic requirements both in time and place.

3.5. Division of Road System by Function

3.5.1. The road system should be divided by functions into different classes but there must be harmony between the different classes of roads in spite of the difference of interest of each class.

3.6. Highway Traffic Characteristics

3.6.1. Traffic is where people are, being greatest where population concentration is the greatest, and each centre has its own zone of influence varying according to its import and export potentialities. Traffic also develops in an area where the development of countryside takes place.

3.6.2. Traffic varies directly with the population and produce of connected towns and villages and inversely as a power of distance, the index of power being one or more.

3.6.3. Highway traffic is one of short range as trip milage is limited to short lengths and concentrated in a small radius round the towns and cities.

3.6.4. Highway traffic is complementary and seldom competitive with rail traffic which primarily serves long distance transportation.

3.6.5. The highway traffic from towns proceeds outwards and from the rural areas towards their marketing centres to feed the needs of the people and transport the produce to the urban areas on commercial scale. The cities and towns are thus the magnets but not independent generators of traffic.

3.6.6. Cities cannot be by-passed on their peripheries. What is required is the distribution of traffic in the city by intersecting circles or rings.

3.6.7. Highway traffic is composed of various speeds and there should be facility for the free flow of traffic. Approach to uniformity of speed is the approach to resolving congestion of traffic.

3.7. Road Design

3.7.1. Improvement of roads should not be slowed down as it will be difficult to make up the lag and road investments will depreciate.

3.7.2. Roads can be built for any vehicle but not for an indefinite vehicle. The factors which influence the design of a road are:

Axle load —on pavements
Gross and axle loads —on bridges

Width	—on lane and pavement width
Height	—on vertical clearance
Length	—on curvature and sight distance
Braking capacity	—on sight distance
Power	—on grades

3.7.3. Road design, vehicle regulation, and vehicle taxation must be in balance as roads are built for vehicles. Vehicles are to be regulated for the capacity of roads that are built. There is great disparity in maintaining this balance in India and greater co-operation is necessary between the different organizations dealing with road construction, regulation of vehicles and taxation of vehicles.

3.8. These fundamentals, if kept in view, would enable the planner to indicate what his plan should be. The minimum milage of the highway system with maximum accessibility bringing maximum benefit and minimum expenditure in construction and maintenance is the goal to be achieved. In India, maximum accessibility may be assessed in relation to dispersal of its villages. This varies from 1 to 4 miles in developed agricultural areas. From various considerations the average accessibility of one mile to any road and of 2 miles to a metalled road may, therefore, be reasonable to adopt at present so as to link up all the villages and towns with a population of 2,000 and over to a metalled road. A lower standard than this is not desirable considering the backwardness of the country in respect of road communications as compared to other countries of the world, and a higher standard, though desirable, would probably not be feasible in view of the limited resources available.

3.9. In a democratic society, the function of the Government is to create conditions to supply the basic framework of efficient material environment in the form of various public services. The development of the highway system is an important requisite for the economic development of the nation and it is up to the Government to finance and execute this nation-building activity from the public sector.

3.10. Development of the highway system being a colossal affair needs to be divided broadly into two categories for purpose of allocating the share of responsibility. The framework of the highway structure, comprising mainly National Highways, State Highways, Major District Roads, and Other District Roads should be allocated to the State's share and the development of the feeder and link roads should appropriately be the

job of both public and private sectors although the latter can as well assist the State by rendering voluntary service for State Roads.

4. EVOLUTION OF A FORMULA AND PATTERN FOR INTEGRATED HIGHWAY SYSTEM

4.1. The Chief Engineers of all the States in India, in their meeting held at Nagpur in December 1943, evolved for the first time a "Grid and Star" formula to evaluate the requirements of different categories of roads with a view to balance the needs of the highway system during the next 20 years. This formula was represented symbolically as follows:

(a) Milage for metalled roads $= \frac{A}{5} + \frac{B}{20} + N + 5T + D - R$

(b) -do- unmetalled roads $= \frac{V}{5} + \frac{Q}{2} + R_1 + 2S + D$

where A represents agricultural area.

B represents non-agricultural area.

S or N represents number of towns and villages in the area in question having a population between 2001 to 5000

T	-do-	-do-	over	5000
V	-do-	-do-	below	500
Q	-do-	-do-	between	501 to 1000
R_1	-do-	-do-	between	1001 to 2000

D represents development allowance assessed at 15 per cent

R represents railway milage in the area under consideration.

4.2. The basis of these formulae is not indicated in the Nagpur Plan as to how they were arrived at and what was their scientific bearing. Obviously, they appear to be purely empirical with arbitrary constants and co-efficients. They also do not indicate any specific pattern for the layout of the road system. It will not be wrong to presume that the selection of roads for post-war schemes has so far been based on different considerations in different States with a wide divergence in some cases. The principles so far adopted may not, therefore, be universally applicable and fundamentally sound.

4.3. Revision of Grid and Star Formula

4.3.1. The rate of development in the country since Independence has been more rapid than anticipated; the targets worked out according to the Grid and Star formula have been found to be far too inadequate for the needs of this country. India's position today in the road statistics of the world is rather disappointing. It is for these reasons that the Chief Engineers of all the States decided to revise the Grid and Star formula in such a manner that the targets for the next 20 years from 1961 to 1981 may be more realistic than before. The revised formulae finalised by the Committee of Chief Engineers in their meeting held in January 1959 at Hyderabad are given in Appendix IV.

4.3.2. These formulae are in one way an improvement over the Grid and Star formulae but they still do not indicate any scientific approach to the problem in the sense that they still contain arbitrary constants and co-efficients. The milage on area basis has now been practically doubled than before but this increase is still not in proportion to the overall increase in the co-efficients of other constants. The targets of road milages that work out for different categories of areas are different with different ranges of accessibility as indicated in Tables 1 and 1 (a).

4.3.3. According to these formulae, the targets of road milage for different regions also differ widely and, in some cases, the average milage per hundred square miles of area works out in excess by about 40 to 50 per cent and, in other cases, much less. This state of diversity can neither ensure equal range of accessibility in the various regions nor parity in development between the different regions.

4.4. Graphical Representation of Grid and Star Formula

4.4.1. In the absence of any indication in the Grid and Star formula as to how it could be made applicable to decide the alignment and layout of the road system, no technical and scientific interpretation can be given to it matching the original interpretation of the designers. However, in view of the first principle of highway planning, i.e., "length of road system is dependent on the form of its layout and vice versa", a graphic interpretation of the original Grid and Star formula is discussed here. This will serve as a foundation for the Crystal Pattern formula proposed in this Paper.

4.4.2. The original Grid and Star formulae for metalled and unmetalled roads were designed independent of each other. The first interpretation that can, therefore, be given to them is that the metalled roads were designed to form the framework of the highway structure and the unmetalled roads as their connecting links and feeders. For further interpretation of the individual formulae, some data has to be assessed for the values of their constants. The data given in Appendix V is adopted in the present calculations.

4.4.3. The milage of metalled roads per hundred square miles of total area, according to the original Grid and Star formula, will work out as under:

$$\begin{aligned}\text{Milage} &= \frac{A}{5} + \frac{B}{20} + N + 5T + D - R \\ &= \frac{81}{5} + \frac{19}{20} + 2.4 + 5 \times 0.271 + D - R \\ &= 16.20 + 0.95 + 2.40 + 1.355 + D - R \\ &= 20.905 + D - R \text{ miles.}\end{aligned}$$

This indicates that the distance between two metalled roads is not to exceed 10 miles as it is then alone that it will be possible to bring towns with a population of 5,000 and above within 5 miles of a metalled road.

4.4.4. Considering the formula for metalled roads only, the structure of the road system can be represented graphically as below within the aforesaid interpretation:

(a) The length of road works out to 20.905 miles per hundred square miles of area excluding consideration of development allowance at 15 per cent, less railway milage at 2.74 miles per hundred square miles of total area (the former has been reduced from 15 to 5 per cent and the latter has been deleted in the revised formula). In an area of 25 sq. miles, the length of the road will, therefore, be 5.226 miles as indicated in Fig. 1.

This will leave gaps of 1.844 miles in between two diagonal lengths to join in a continuous line and these will be located at a distance of 3.535 miles apart in parallel lines.

(b) If out of every three such parallel rows two rows of these strips are removed, it will increase the distance between them to 10.605 miles which is near the maximum permissible distance as indicated above. These spare lengths of the two rows can be utilised to fill up the gaps of every third row and for placing ordinates or perpendiculars on the

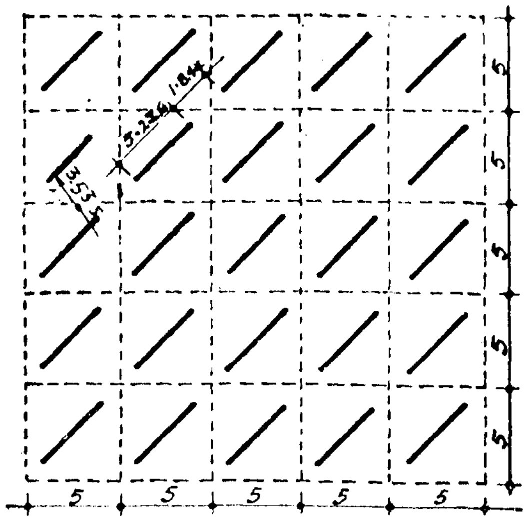


Fig. 1

through roads so formed, so as to join them with cross roads at certain distances as indicated in Fig. 2.

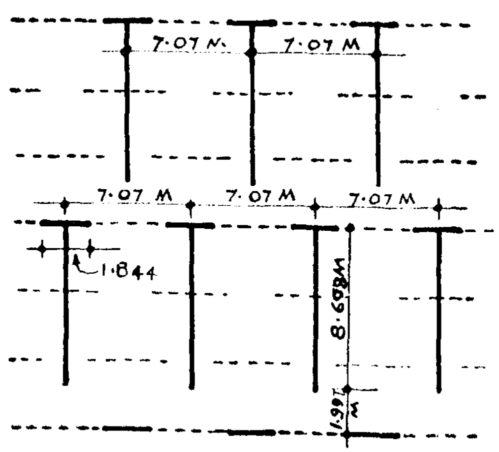


Fig. 2

The length of the two rows removed is 10.452 miles. Opposite each gap of third row, after utilising 1.844 miles for the gap, it leaves 8.608 miles for the perpendicular ordinates which are thus distanced 7.07 miles apart. These ordinates fall short in length by 1.997 miles.

(c) If the distances of these ordinates are increased to 10.605 miles as against 7.07 miles distance of the parallel roads, it will save one perpendicular length out of every three, which will be utilised, as given in Fig. 3, to extend a length of 1.997 miles in each of the other two and will still leave 4.614 miles spare.

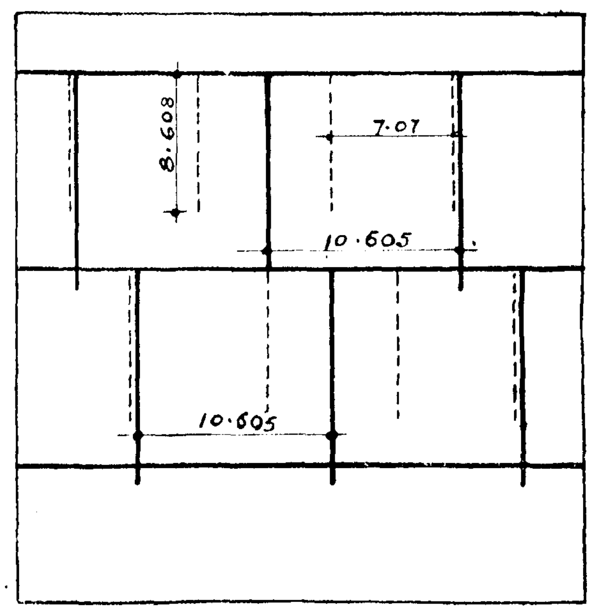


Fig. 3

(d) This spare length can be utilised as perpendicular to the sides in a length of 1.154 miles on all the four sides as indicated in Fig. 4. This figure indicates the layout of roads such that no place will be more than 5 miles from a metalled road. This pattern is still defective in the sense that one has to travel along the opposite end and thus it will detour for about 40 per cent of the actual distance and will be uneconomical in transport.

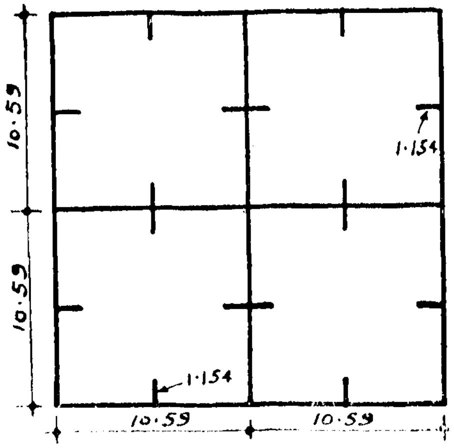


Fig. 4

4.4.5. However, this form is still capable of further improvement so that it could give minimum detour. From Plate I, giving the typical layout of the road system as naturally developed in the countryside, it is observed that roads radiate from each village in all directions, the number of radiating roads depending upon the potentiality of each village and its population. These radiating roads linked together form periphery roads, and a pattern of radiating and circumferential roads similar to a spider's web is observed to have developed, depending upon the conveniences and natural tendencies of the people but without any scientific plan or foresight as to how these tracks will adjust in the future trend of development of the highway system. It is for this reason that some of these tracks cannot be developed at this stage and some of them will need improvement in their alignment to be more useful by economic considerations.

4.4.6. If this form of layout of the road system is adopted with some improvement to give a maximum accessibility of 5 miles, the length of detour can be eliminated greatly. To design such a pattern of road system, a form given in Fig. 5 is indicated by trial and error method after calculating the milage for different patterns.

4.4.7. This specific pattern of layout of road system requires 396.96 miles of metalled roads for an area of

1,794.37 sq. miles. According to the Grid & Star formula, the milage of roads for this area works out to $1{,}794.37 \times \frac{20.905}{100}$ = 375.03 miles only. A gap of 396.96 − 375.03 = 21.93 miles or

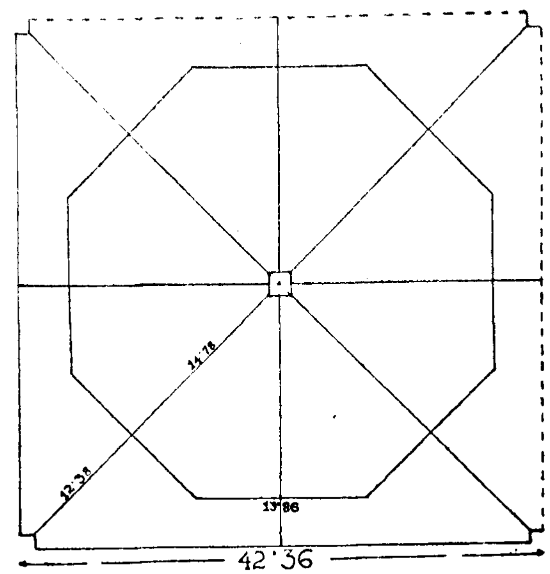


Fig. 5

1.20 miles per hundred sq. miles of area, or about 5.52 per cent of the milage, is still required. This can be covered with advantage by the development allowance of 15 per cent as originally assumed. This pattern will have radial roads in all directions with a maximum accessibility of 5 miles and will be an economical layout for purposes of transport. It is not obligatory that the roads should be built along all these alignments at one time because some alignments may have to be altered or deferred due to unfavourable physical features of the country or some other reason. Thus the milage with this pattern will not exceed the calculated milage of the formula.

4.4.8. In the formula for unmetalled roads, the maximum distance provided for any village with a population of 2,000 to 5,000 is only 2 miles, meaning thereby that such villages shall not be more than 2 miles away from a metalled road. But in case of other villages, the milage proposed is very much less, i.e., $V/5$, $Q/2$, R, etc. This means one mile for every five villages having a population below 500, one mile for every two villages having a population between 500 to 1,000 and one mile for every village having a population between 1,000 to 2,000.

4.4.9. The villages of different population up to 10,000 are spread over generally at a distance of ½ to 8 miles—the small villages being dispersed at ½ to 4 miles apart. Obviously, therefore, the Grid and Star formulae, both original and revised, are not suitable in their present form for a direct approach to the present and future problem of highway development and a more realistic and rational approach is required to balance the needs of the country at the present rate of progress for the next 20 years.

5. CRYSTAL PATTERN FORMULA

5.1. Road milage in an area depends upon the form of layout of the road system in that area, the layout in turn depending upon the desired accessibility to the road system from any point within the area. This indicates that it is the accessibility to a road which should be assessed first. But, for a general appraisal, it is better to establish a general relationship between the accessibility to a road and the road milage that would be required with a certain pattern of layout of the road system such that no road once built is lost at any later stage. There can be different patterns to meet this requirement but the one that has been found more suitable is discussed here.

5.2. Accessibility-Milage Ratio

5.2.1. This relationship is represented in the accessibility-milage ratio chart given in Plate II. The curve seems to be parabolic but it is not so, as it does not show a point at which the characteristics reverse. Further, it is not part of a parabola or an ellipse as negative values for accessibility or increase in targets in reverse direction are impossible. Secondly, at two points, viz., when accessibility is zero and when the increase in milage target is zero, there are two asymptotes and at these points a part of the curve tends to infinity. A peculiar feature of this curve is that the differences in the increase of milage target for corresponding decrease in accessibility in arithmetical progression are in geometrical progression with a common ratio of 1.5 up to accessibility of 5 miles; thereafter, it is in arithmetical

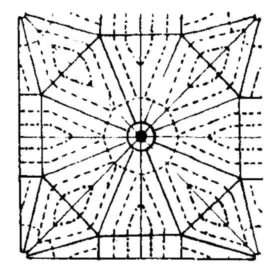
progression with a common difference of one up to 10 miles accessibility. A similar range of differences in milage targets is observed when the accessibility increases from 10 to 50 miles and 50 to 100 miles respectively. However, this curve gives a varying range of milage targets for different values of accessibility and enables a designer to read the milage target for any required accessibility direct from the curve. Road milages dispersed according to this pattern for different values of accessibility, Figs. 6 to 15, are found to be in agreement with the results obtained from this curve. This indicates that this pattern is not only elastic but also permits of stage development where no road constructed according to this system will be lost later on as the system grows and the figure of accessibility on the same pattern decreases.

5.2.2. The geometric pattern, designated as "Crystal Pattern", with 1 mile accessibility for evaluating road milages of different classes of roads is given in Plate III.

CRYSTAL PATTERN

DISPERSAL OF ROADS FOR VARIOUS ACCESSIBILITIES.

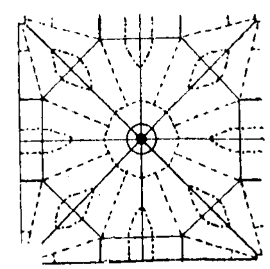
664 MILES TO 687 MILES
‰ SQ.MILES



ACCESSIBILITY - 1 MILE

Fig. 6

465 TO 485 MILES
‰ SQ MILES



ACCESSIBILITY - 2 MILES

Fig. 7

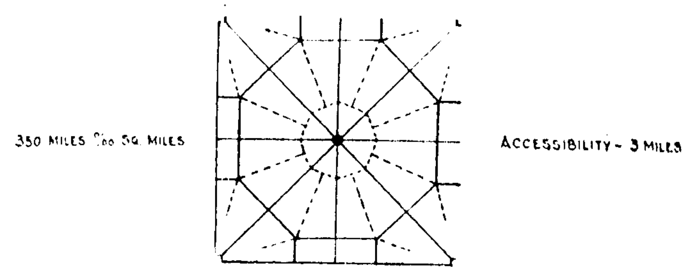


Fig. 8

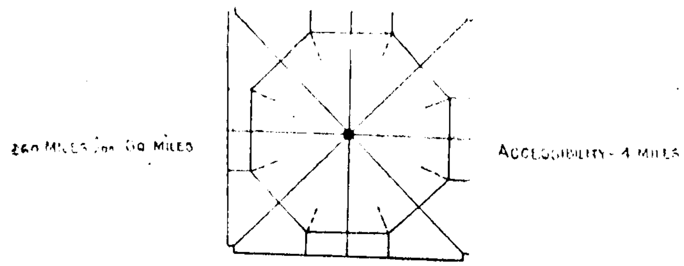


Fig. 9

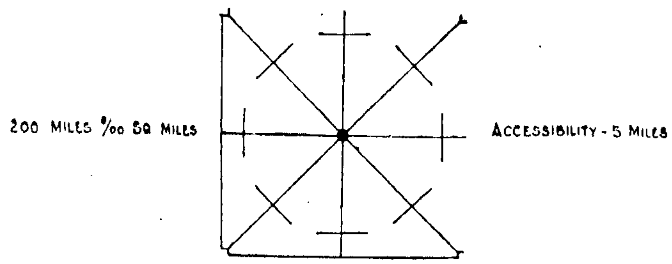


Fig. 10

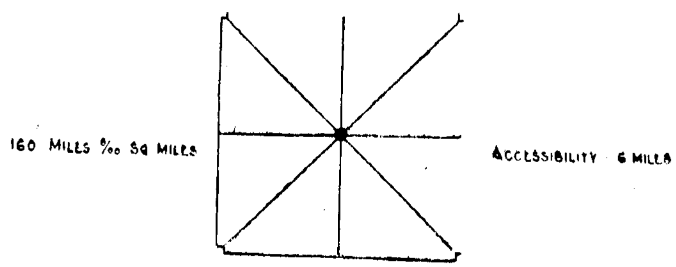


Fig. 11

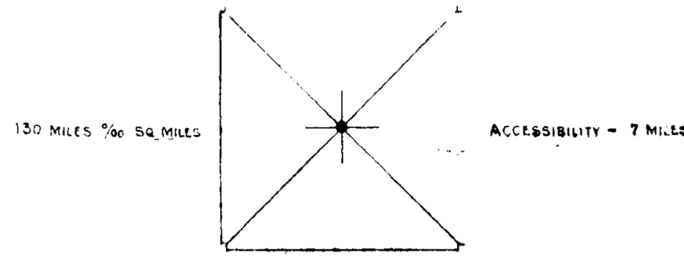


Fig. 12

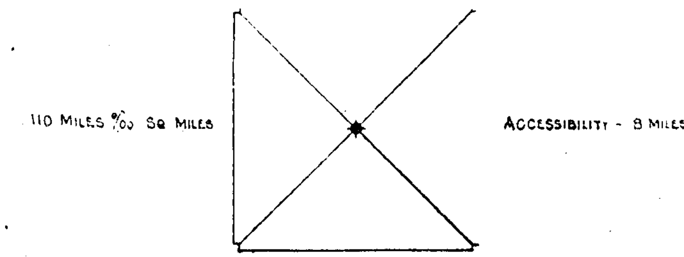


Fig. 13

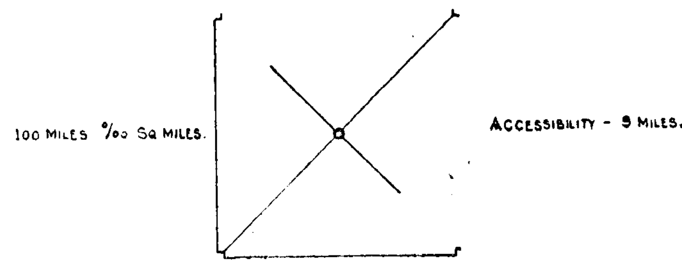


Fig. 14

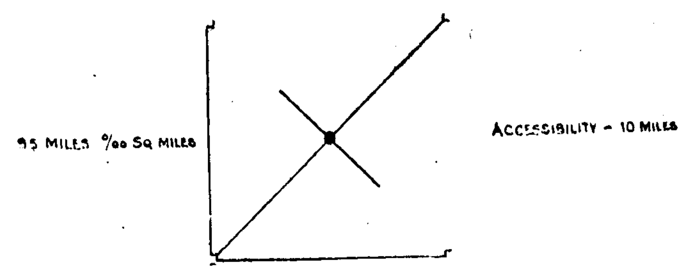


Fig. 15

5.3. The area of one grid is assumed as 1794.37 sq. miles, with a side of 42.36 miles, and the road system is designed along radial and circumferential alignments. No place in this pattern shall be more than one mile from an unmetalled road and 2 miles from a metalled road. The total milage of the road system will be about 66.4 miles per 100 sq. miles of total area with a facility to restrict the milage to any lower limits with increased accessibility figure.

5.4. **Location of Nucleus Centres and Their Field of Influence**

5.4.1. Since the crystal pattern is only an amplification of the Grid and Star formula, the location of the stars, known as nucleus centres, is an important factor. Every nucleus centre has its zone of influence and will develop radiating roads within this zone. For economic development of the country, therefore, it becomes necessary to restrict the location and development of nucleus centres in such a manner that the field of influence of one does not impair or adversely affect that of the other and overlapping is avoided. The field of influence judged for each village and town of different potentiality, with a slight margin of error on the favourable side, is suggested below:

			Field of influence	
(i)	Towns and villages with a population between	2000-5000	About 2 to 3 miles on all sides	C
(ii)	-do-	5000- 10000	,, 3 to 4 -do-	B
(iii)	-do-	10000- 50000	,, 4 to 6 -do-	A
(iv)	-do-	50000-100000	,, 6 to 8 -do-	A
(v)	-do-	100000 and over	,, 8 to 12 -do-	A

5.4.2. The range of influence may be said to increase in a geometrical progression with a common factor of 1.5 for graded population depending upon the importance and potentiality of the place and facilities of transport available. For purposes of highway planning, the nucleus centres may be classified under categories 'A', 'B', and 'C' as indicated above. Class 'A' may be district headquarters with the exception of hill districts; Class 'B' tehsil headquarters or a district headquarter in hilly region; and Class 'C' treated as a local centre in rural areas. These centres will be fed by all the villages of smaller population around them and each centre, in turn, will feed the other centre of greater potentiality and importance or the next above that depending upon the proximity.

5.5. **Areas of Different Categories**

5.5.1. When the object is to develop the entire countryside so as to increase its resources both in respect of agricultural and

industrial production, no area should be ignored or given low priority for development because it is an undeveloped and thinly populated area. Such areas should rather be given higher priority as they would not only increase production but would also assist in providing accommodation for rehabilitation of increasing population. Areas of this type are, in fact, not isolated but are generally pockets in developed areas except in case of hilly regions and the State of Rajasthan. The principle of providing a lower range of milage for semi-developed areas, i.e., B/5 or 50 per cent of developed areas, is not a fair treatment. The same consideration should be given to semi-developed areas as for developed areas for the reasons that, firstly, the areas of different categories are not located independently and separately in the country but they are generally interwoven with developed areas in such a manner that they cannot be separated one from the other. Secondly, even if they are isolated, they are possibly the unexplored treasures of the nation's wealth.

5.5.2. There could, however, be some justification to treat the areas on geographical considerations and to divide the into the following categories:

(a) Hilly regions

(b) Semi-hilly regions

(c) Plains:

(i) Fertile

(ii) Unculturable and barren.

The pattern of road development in either case shall be the same as discussed heretofore, i.e.,

(a) In case of hilly regions, areas above 10,000 ft altitude are generally not inhabited but for strategic considerations road construction is essential in certain reaches. In such areas the milage of the road system can be reduced by 50 per cent only.

(b) In semi-hilly regions, no deduction in road milage is advisable.

(c) In plains, which are barren or desert land, the need for full length of road milage may not be immediately necessary but the target should be kept on the same basis. Construction may be limited to through routes and essential feeders to begin with and further constructions could follow as need arises, but the system should not develop in a haphazard manner.

5.6. Classification of Highways

5.6.1. The intensity and nature of traffic on the highways varies according to their importance and it is necessary that the system of roads should be grouped under different classes. These can be assessed on the basis of administrative and other considerations and the importance of each road. Broadly speaking, the system of roads can be divided into four groups :

(i) **Group I**—Super Primary road—system to comprise inter-State system of roads-National Highways, Express Highways and State Highways.

(ii) **Group II**— Primary road system—to comprise Major District Roads.

(iii) **Group III**— Secondary road system—to comprise metalled and unmetalled Other District Roads which may be upgraded in subsequent phases and form part of the main highway structure.

(iv) **Group IV**— Feeder road system—to comprise unmetalled or metalled feeder roads from and to smaller villages or villages which are not on the main roads, and roads for interlinking villages for social and other considerations.

5.7. Description of Highways

5.7.1. (a) **Group I—Super Primary Road System**

(i) **National Highways**— These shall be the main arterial routes throughout the length and breadth of the country connecting State capitals, important industrial, commercial and trading centres, places of national importance, and also such roads as are required for strategic considerations.

(ii) **Express Highways**— In the heavily congested and highly industrialised areas, it will be necessary to provide extra parallel lengths of roads specifically meant for uninterrupted flow of heavy and fast through traffic. Such highways will have level junctions with National Highways and State Highways near important towns while all other types of roads will be crossed by over or under-bridges at their crossings. These being supplementary roads running parallel to the highway system do not find place in the pattern and are not to be accounted for in the target of road milage.

(iii) **State Highways**— These will be main highways similar to the National Highways and will interconnect district headquarters within a State and in the adjoining States.

(b) **Group II—Primary Road System**

(i) **Major District Roads**— All the roads radiating from the star in a grid other than National Highways and State Highways and those forming the framework of the highway system along periphery roads shall be known as Major District Roads. They shall supplement the State Highways for interlinking main centres of development to sub-centres in the region of their grid as well as in the adjacent grids.

(c) **Group III—Secondary Road System**

(i) **Other District Roads—** All roads running parallel to the Major District Roads shall be classified as Other District Roads and alternate ones will be metalled in the first phase, the rest to be metalled in subsequent phases. These shall also form part of the integrated highway system.

(d) **Group IV—Feeder Roads**

These roads shall be small village roads or feeder roads to main roads connecting villages together or to the main highways. They shall be unmetalled or metalled depending upon the nature of traffic they carry. They shall be constructed and maintained by the Gram Panchayats or Antarim Zila Parishads being of local importance only.

5.8. Targets of Road Milage

5.8.1. According to the Crystal Pattern of highway system, roads radiate from the star in a grid and the Group I roads, i.e., Super Primary Roads, shall comprise these roads as they are through roads. There are two sets of roads which radiate from each star, one set of diagonal roads and the other of cross roads. The length of the diagonal roads in one grid of 1,794 sq. miles is 6.7 miles per hundred square miles and of cross roads 4.7 miles per hundred square miles. Since the average condition for the whole of India cannot be applicable to Uttar Pradesh and similar regions, the targets for such regions will have to be worked out separately and the maximum of the limits obtained adopted for developed and agricultural areas like Uttar Pradesh. Table 2 indicates the values of constants for different accessibilities to be adopted for evaluating road mileage.

5.8.2. (a) **Group I — Super Primary Roads**

(i) **National Highways**: In the alternate design of the layout of main highways, the spacing between two National Highways can be selected either 30, 42.4, 60, 85 or 120 miles. The lower limits of 30 and 42.4 miles may be considered suitable for State Highways as they will connect district headquarters which are generally spaced at such distances in agricultural areas like Uttar Pradesh. The limit of 85 and 120 miles can be suitably adopted for National Highways to suit well developed areas and thinly populated areas respectively. To provide an even distribution of the highways with a spacing of 85 miles, the target for National Highway milage works out to 2.35 miles per 100 sq. miles, and with a spacing of 120 miles apart it works out to 1.7 miles per 100 sq. miles. Thus the former can be reasonably adopted in well developed areas. For evaluating the milage for the country as a whole, an average of these two figures, i.e., 2.03 miles per 100 sq. miles, will serve the purpose reasonably well at present.

The length of National Highways will, therefore, work out to:

(1) For India $12{,}66{,}900 \times \frac{2.03}{100} = 25{,}718$ miles

By adding 5 per cent for development = 27,000 miles

(2) For Uttar Pradesh $1{,}13{,}409 \times \frac{2.35}{100} = 2{,}665$,,

By adding 5 per cent for development = 2,800 ,,

(ii) **Express Highways**: These highways have no place in the design of the pattern as already discussed, but being essential bye-roads for fast through traffic, a milage at 0.45 miles per 100 sq. miles of area or 5 per cent of the milage of National and State Highways may be provided for them. The total length for such highways in India may be assessed at 5,800 miles and in Uttar Pradesh it may be 510 or, say, 500 miles.

(iii) **State Highways**: Out of the radial roads, if the National Highways are selected along diagonal roads at 1.7 miles per 100 sq. miles of area and located at a distance of 120 miles apart, the remaining diagonal roads, i.e., 5.0 miles per 100 sq. miles, shall be treated as State Highways. Where National Highways are laid out along the cross roads of the Crystal Pattern in the grid, the full length of diagonal roads, i.e., 6.7 miles per 100 sq. miles, will be taken for evaluating mileage of State Highways. For total length of State Highways in India, the average figure is 5.85 miles per 100 sq. miles and should be adopted.

(1) For India $12{,}66{,}900 \times \frac{5.85}{100} = 74{,}114$ miles

Adding 5 per cent for development = 3,706 ,,

= 77,820 ,,

(2) For Uttar Pradesh $1{,}13{,}409 \times \frac{6.7}{100} = 7{,}598$,,

Adding 5 per cent for development, say = 380 ,,

= 7,978 ,,

Say 8,000 ,,

(b) **Group II—Primary Road System**

Out of the two sets of radial roads of the pattern discussed for Group I roads, in one case all the four cross roads, the length of which is 4.7 miles per 100 sq. miles of area are, left over and, in another case, half of this length, i.e., 2.35 miles, is left over to be classified as Major District Roads. In addition, there are other radial and circumferential roads in the pattern which form the primary framework of the road system and their length is 25.65 miles per 100 sq. miles of area. The basis for overall average milage of Major District Roads for India will, therefore, be $\frac{4.7 + 2.35}{2} + 25.65 = 29.2$ miles per 100 sq. miles of total area and their length for India will be $\frac{12{,}66{,}900 \times 29.2}{100} = 3{,}69{,}935$, say, 3,88,432 miles including 5 per cent development allowance. For Uttar Pradesh, $25.65 + 2.35 = 28$ miles per 100 sq. miles is to be adopted and the length will be $\frac{1{,}13{,}403 \times 28.0}{100} = 31{,}755$ miles, say, 33,343 miles including 5 per cent development allowance.

(c) **Group III—Secondary Road System**

Other District Roads (O.D.Rs.): As discussed before, this type of roads shall be both metalled and unmetalled. The total milage of O.D.Rs. in the Crystal Pattern for one mile accessibility works out to 29.3 miles per 100 sq. miles of the grid. Out of this, about 9 miles per 100 sq. miles shall be metalled in the first instance and the remaining 20.3 miles will be considered for metalling when the accessibility figure from a metalled road is proposed to be reduced to one mile.

Length of Other District Roads	Milage including 5 per cent development allowance		
	Metalled	Unmetalled	Total
in India	1,19,722	2,70,040	3,89,762
in Uttar Pradesh	10,717	24,173	34,890

5.8.3. The results are summed up in Tables 3 and 4.

5.9. **Advantages of Crystal Pattern Formula**

(a) This formula is based on the layout of the ultimate picture of a road system in the entire area that shall give maximum accessibility to a road system with minimum milage to construct and maintain. It is not based on arbitrary constants susceptible to changes.

(b) No place in the Crystal Pattern layout of roads will be more than one mile from a road or 2 miles from a metalled road within the target of 66.4 miles per 100 sq. miles of total area, whereas the Revised Grid & Star Formula (1959) claims an accessibility of 1.5 miles from any road and 4 miles from a metalled road only in developed areas and in other areas the figure of accessibility increases to 8 and 12 miles from a metalled road.

(c) The formula is simple to understand and easy to handle.

(d) The formula is elastic in the sense that it visualises the ultimate picture capable of reduction according to circumstances and can be adopted for different ranges of accessibilities without difficulty.

(e) It will ensure a direct approach with minimum detour.

(f) It will ensure approach to every area with equal facility.

(g) The layout of the road system will be of the form developed naturally so far and will accommodate the existing system to a great extent to fit in the pattern with minimum wastage.

(h) The big towns will be directly connected to each other without any extra feeder roads.

(i) The feeder roads necessitated will be reduced to a minimum and will generally be of lower grade.

(j) What is built will not be lost.

6. MASTER PLAN-PREPARATION AND PHASING OF THE ROAD SYSTEM

6.1. There exists a great disparity in the dispersal of population not only in different types of areas in different States but also amongst their district units. It will, therefore, be advisable to group a few districts together to design their combined road system. A single district can be considered independently but the results may not be very satisfactory. The road system should always be designed for a district in conjunction with its adjoining districts.

(a) **Group I — Super Primary Roads**

For designing this system of roads for each State, it is advisable to consider each State along with portions of the adjoining States so that selection and alignment of these highways may be in continuity.

(b) **Group II — Primary Road System**

The roads of this system continue from one district to another and cannot, therefore, be designed independently for each district. They should likewise be designed for every district along with its adjoining districts to maintain continuity of the system.

6.2. To prepare a Master Plan of the road system for any district or group of districts, the *first step* according to fundamentals of highway planning is to decide the accessibility aimed at by the system. For the desired accessibility, the target of probable milage of roads per thousand square miles of area may be obtained from the Accessibility-Mileage Ratio Chart (Plate II). This figure when multiplied by the total area under consideration will give the total milage required for that area, Next, a map of the area should be prepared to a scale of 1 in. = 2 miles and it should be numbered as map 'M' (M_1 & M_2) (Plate IV) for future reference. This map should contain all the information as discussed in para 2.2.2 and mentioned in Appendix III. In case it is not possible to show this information on one map, it may be shown on two or three maps so that each set of information is intelligible to the designer.

6.3. The *second step* is to place a tracing paper on one of the aforesaid maps depicting all the villages, towns, roads, railways, rivers, etc., and to trace the existing system of roads on this tracing paper. This will be known as map 'P' (Plate V). The roads proposed by different agencies shall be marked in pencil on this tracing paper so that they may also be considered while drawing up the Master Plan.

6.4. In designing the Master Plan, the following considerations shall be kept in view:

(a) The layout of the road system shall, as far as possible, be developed in the form approaching the "Crystal Pattern," though it may not be exactly in the same geometrical form, depending upon the geographical features of the countryside, etc.

(b) All the towns and villages above 2,000 population in plains, above 1,000 in semi-hilly areas and 500 in hilly regions shall, as far as possible, be connected directly by the Super Primary or Primary road system.

(c) All industrial and commercial centres, places of pilgrimage, historical places, places of excursions for tourists and health resorts, places of strategic importance, tehsil and police headquarters shall also have to be connected and inter-connected by the metalled road system in each grid.

6.5. The road system should be developed on the tracing paper, map 'P' (Plate V), keeping in view the above considerations as well as the physical features and topography of the country. For selecting alignments, existing tracks as indicated on 1 in. = 1 mile scale map during the course of factual survey will be taken as a guide. It will be observed that the existing metalled roads and the demands of different agencies will automatically be adjusted with the development of this pattern if it is worked up a little intelligently. The layout should be checked at least twice after it is designed and it will be observed that the plan is greatly improved in checking and rechecking. It needs a little patience and judicious thought to select the correct layout of the road system even though the pattern may be before the designer. The selection of roads should be done with all the possible considerations in view.

6.6. The *third step* is to check the system to see if it conforms with the pattern. If not, such roads should be rechecked which deviate from the aforesaid principles and it will be found that, unless there is any serious natural impediment, these layouts are susceptible of correction to accord with the pattern. The classification of roads may then be decided in accordance with the pattern to give relative importance to each road by virtue of its location and utility. While fixing the classification, the alternative designs for the National Highway system should also be considered as it is not necessary that the National Highways should pass through each star of all the grids.

6.7. The *fourth step* is to test each road in the system for its utility per mile with respect to population and productivity of the area where the road lies. The total area can be divided into different sectors to facilitate tabulating the roads for testing their utility. This is done as follows.

6.7.1. On the Master Plan, Map 'P', place a tracing paper and on this paper trace the designed road system with different classifications. This will be map 'R' (Plate VI). Mark the zones of each road with dotted lines drawn parallel to each road at a distance of the required accessibility for which the system has been designed. Towns and villages lying within the zone of each road on either side shall determine its utility. Place map 'R' on original map 'M' on which population is marked, so that the existing roads and boundaries of the one coincide with those of the other. The villages and towns of different population lying within the zone of each road in each sector may be listed separately under their respective columns in Table 5. The villages of 2,000 to 5,000 population in plains, 1,000 to 5,000 in semi-hilly regions and 500 to 2,000 in hilly regions may be treated as one unit. Those with a population below 2,000 in plains, below 1,000 in semi-hilly regions and below 500 in hilly regions may be treated as half a unit. Those between 5,000 to 50,000 population may be treated as 2 units and above 50,000 as 4 units. The number of units served by each road shall accordingly be calculated and recorded in the Table. The total number of units served per mile shall be determined and noted in the required columns. On relative comparison of their utility, the priorities for each road can be allotted in this order and noted in the column for priority.

6.8. Similarly, the total productivity served by each road should be worked out by placing map 'R' over the original map 'M' on which the productivity is marked. This shall be calculated by multiplying the zonal area of each road for each of its productivity by the productivity in maunds or tons per acre or per square mile for that area and recorded in Table 5. The productivity served per mile and the priority in relation to this may likewise be tabulated. This process should be done for metalled and unmetalled roads separately and combinedly to indicate their relative merits and priorities. Administrative considerations will give weightage to a road for increasing its priority even though it may have a lower priority as regards its utility. This consideration will apply in case of strategic or administrative routes only.

6.9. Phasing of the Road System

6.9.1. The results of Table 5 for metalled and unmetalled roads are plotted to a scale on Figs. 16 and 17 for comparative study of population and productivity per mile separately of the

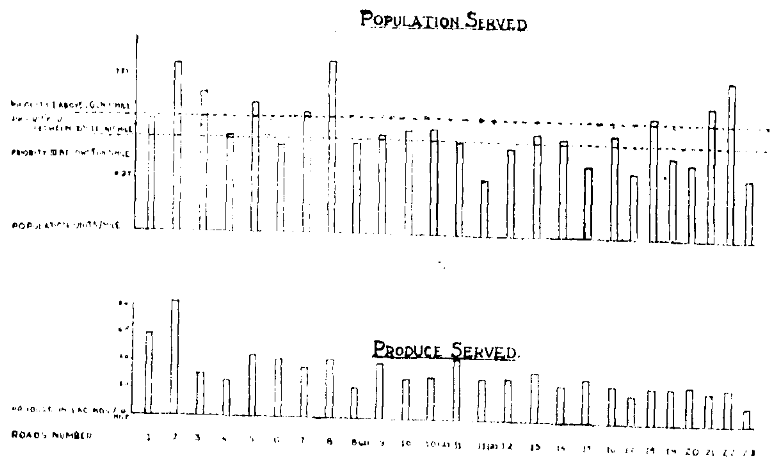


Fig. 16

Comparative study of population & productivity served for individual new metalled roads in Meerut district

entire road system at a glance. By drawing a line across the population utility ordinates for each metalled road at a stage of one unit per mile, the ordinates that stand projected above this line will indicate a higher priority for the respective roads and

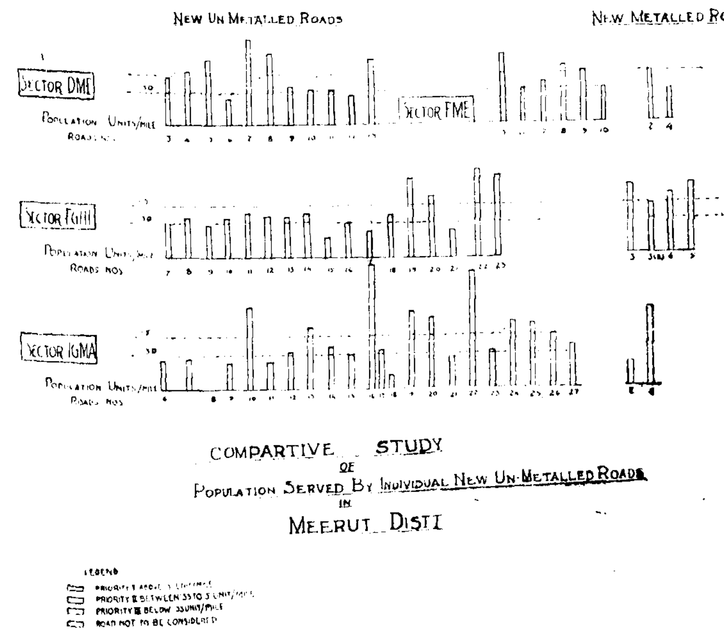


Fig. 17

can be grouped in Phase I. The relative height of these ordinates above this line will indicate their relative priorities.

6.9.2. Similarly, when a line is drawn across these ordinates for a utility of 0.75 unit per mile, the ordinates projecting above this line will indicate roads which should fall under Phase II and their relative heights will indicate their relative priorities. Likewise, the process can be repeated by drawing lines at different stages of utility, say, 0.5 and 0.25 unit per mile for Phases III and IV respectively.

6.9.3. The above process will facilitate the selection of individual roads on their real merits and on a rational basis. It will be advisable to spread the development of roads in each sector with a view to utilise local labour and provide employment. Since road construction is a continuous process and is done in stages, if in any sector a road is not justified as a metalled road in the first phase it may be taken up as an unmetalled road, provided it is justified as such with sufficiently high priority.

6.9.4. A similar process is to be adopted in respect of unmetalled roads but in their case the unit standards for utility have to be downgraded for each phase as noted below :

			Unit per mile and above
For Phase I	the utility standard will be		0.5
-do- II	-do-		0.3
-do- III	-do-		0.2
-do- IV	-do-	below	0.2

The proposed metalled roads which do not get high priority amongst themselves should again be considered along with the unmetalled roads, and if they get higher priority amongst them they should be selected for stage development in preference to unmetalled roads and constructed as unmetalled roads in Phase I. This will facilitate selection of roads in each sector and their development will be spread all over the area in each phase.

6.9.5. The roads under each phase shall be tabulated with their costs in the order of their relative priority. The cost of minor bridges and culverts on such roads shall be included in the cost of the roads. Major bridges above 100 ft span should, however, be shown separately for each road so that they can be taken up either along with the construction of the road or subsequently at a later stage.

6.9.6. Each phased programme shall include all types of works, i.e., reconstruction, improvements, new construction of roads and bridges and buildings, tools and plant required in relation thereto and other miscellaneous items. The programme may be divided under the following sub-heads :

A. Reconstruction and improvements
B. New road construction
C. Major bridges
D. Miscellaneous works

6.9.7. Reconstruction and improvement are essential in those cases where traffic intensity has increased and the existing surface cannot be maintained economically for the increased traffic. The traffic conditions may necessitate upgrading or widening of the traffic lane which shall be provided for according to the needs of traffic.

6.9.8. New construction of roads will include both metalled and unmetalled roads. The construction of these roads should be in accordance with their relative priorities.

6.9.9. **Major bridges** : All unbridged crossings on existing roads shall be given preference in fixing priority if other conditions are similar and those on new roads can be taken up along with the construction of roads or immediately thereafter.

6.9.10. **Miscellaneous items** : Under this sub-head provision for requirements of buildings such as inspection houses, godowns and other ancillary buildings, requirements of tools and plant and machinery and implements for research work should be made.

6.9.11. The Master Plan so prepared and phased will be a rational and economical plan based on scientific principles. A sample Master Plan explaining the different processes as detailed in the foregoing paras is illustrated in maps 'M', 'P' and 'R', (Plates IV, V & VI.)

ACKNOWLEDGMENT

The Author thanks Shri M.S. Bisht, C.E., MIE., MASC., now Member, Public Service Commission, Uttar Pradesh, for his kind encouragement and advice and Shri Jagdish Prasad, C.E., MIE., Chief Engineer, U.P., P.W.D., for his kind appreciation and permission to submit this Paper to the Indian Roads Congress for publication.

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TABLE 1

Statement showing requirements of road milages for India on the basis of revised grid and Star Formula, 1959

	Milage for area			Milage for towns			Total	Grand Total
	Developed area	Semi-developed area	Undeveloped area	Developed area	Semi-developed area	Undeveloped area		
(i) National Highways, State Highways & Major Distt. Roads	1,52,140	25,962	16,438	40,838	3,188	1.341	2,39,907	2.51,902
						Add 5 per cent	11.995	
(ii) Other Distt. Roads & Village Roads (Classified)	1,52,141	25,962	16,439	1,65,589	17,657	8,576	3.86.364	4,05,682
						Add 5 per cent	19.318	
Total:	3,04,281	51,924	32,877	2,06,427	20.845	9.917	6,26,271 31.313	6.57.584

	Total milage	Per 100 sq. miles	Maximum distance from a road
(i) Agricultural and developed area	5,36,243	70	1.43 say 1.5 miles
(ii) Semi-developed area	76,407	30	3.33 say 3.0 miles
(iii) Undeveloped area	44,934	19	5.3 say 5.0 miles

TABLE 2

Values of constants for different accessibilities for 100 sq. miles of area

Classification of roads	For 1 mile accessibility			For 2 miles accessibility			For 3 miles accessibility		
	With National Highways dispersed 120 miles apart	With National Highways dispersed 85 miles apart	Average	With National Highways despersed 120 miles apart	With National Highways dispersed 85 miles apart	Average	With National Highways dispersed 120 miles apart	With National Highways dispersed 85 miles apart	Average
Group I									
(i) National Highways	1.7	2.35	2.03	1.7	2.35	2.03	1.7	2.35	2.03
(ii) State Highways	5.0	6.7	5.85	5.0	6.7	5.85	5.0	6.7	5.85
Total	6.7	9.05	7.88	6.7	9.05	7.88	6.7	9.05	7.88
Group II									
Major District Roads	30.4	28.05	29.22	17.1	14.75	15.92	17.3	14.95	16.12
Group III									
O.D.Rs. (Metalled)	9.0	9.0	9.0	9.0	9.0	9.0	—	—	—
O.D.Rs. (Unmetalled)	20.3	20.3	20.3	13.7	13.7	13.7	11.0	11.0	11.0
Total Groups I, II & III	66.4	66.4	66.40	46.5	46.5	46.5	35.0	35.0	35.0
Extra unmetalled roads	2.3	2.3	2.3	2.0	2.0	2.0	—	—	—
Total	68.7	68.7	68.70	48.5	48.5	48.5	35.0	35.0	35.0

TABLE 3

Abstract of road milages for different classifications of roads according to Revised Grid and Star Formula 1959 and Crystal Pattern Formula inclusive of development allowance at 5 per cent

Classification of roads	With accessibility of 3 miles from a metalled road and 1.5 to 1.75 miles from any road in developed area, with accessibility of 5.5 miles from a metalled road and 3 to 4 miles to any road in semi-developed area, with accessibility of 9.0 miles from a metalled road and 5 to 6 miles to any road in undeveloped area				With accessibilty of 1 mile to a road in developed area, 3 miles to a road in semi-developed and 5 miles to a road in undeveloped area		With accessibility of 1 mile from any road and 2 miles from a metalled road throughout	
	As per Revised Grid and Star Formula 1995		As per Crystal Pattern Formula		As per Crystal Pattern Formula			
	India	Uttar Pradesh	India	Uttar Pradesh	India	Uttar Pradesh	India	Uttar Pradesh
	Miles	Miles	Miles	Miles	Miles	Miles	Miles	Miles
Group I								
(i) National Highways	31,700	3,000	27,000	2,800	27,000	2,800	27,000	2,800
(ii) State Highways	70,300	6,600	77,820	8,000	77,820	8,000	77,820	8,000
Group II								
(iii) Major Distt. Roads	1,49,902	15,600	2,50,160	21,920	3,08,760	27,280	3,88,432	33,343
Group III								
(iv) Other Distt. Roads								
(a) Metalled	1,77,681	23,500	71,890	6,400	71,890	6,400	1,19,722	10,717
(b) Unmetalled	2,28,002	32,900	1,65,770	14,000	1,92,230	16,350	2,70,040	24,173
Total metalled :	4,29,583	48,700	4,26,870	39,120	4,85,470	44,480	6,12,974	54,860
Total unmetalled :	2,28,002	32,900	1,65,770	14,000	1,92,230	16,350	2,70,040	24,173
Grand Total :	6,57,585	81,600	5,92,640	53,120	6,77,700	60,830	8,83,014	79,033

TABLE 4

Statement showing comparative figures of road milages with different accessibilities according to Revised Grid and Star Formula 1959 and Crystal Pattern Formula

	India				Uttar Pradesh			
	Metalled road milage	Un-metalled road milage	Total road milage	Rate per cent sq. mile	Metalled road milage	Un-metalled road milage	Total Road milage	Rate per cent sq. mile
I. To achieve the accessibility of :								
(i) 2.0 miles from a metalled road and 1.5 to 1.75 miles from any road in developed area;								
(ii) 5.5 miles from a metalled road and 3 to 4 miles from any road in semi-developed area;								
(iii) 9.0 miles from a metalled road and 5 to 6 miles from any road in undeveloped area.								
Milage required								
(a) According to Revised Grid and Star Formula 1959	4,29,583	2,28,002	6,57,585	51.9	48,700	32,900	81,600	72
(b) According to Crystal Pattern Formula	4,06,540	1,57,880	5,64,420	44.6	37,260	13,330	50,590	44.6

Note : 1. Distribution of total area in 3 categories, i.e., developed, semi-developed and undeveloped area in

	Developed	Semi-developed	Undeveloped
India ...	60.5 per cent	20.5 per cent	19.0 per cent
Uttar Pradesh	59.7 ,, ,,	14.7 ,, ,,	25.6 ,, ,,

2. The milages according to Crystal Pattern Formula are exclusive of any development allowance as the same is worked out on the basis of accessibility. The development allowance will be extra.

(*Contd. on next page*)

TABLE 4 (*Contd.*)

	India				Uttar Pradesh			
	Metalled road milage	Un-Metalled road milage	Total road milage	Rate per cent sq. mile	Metalled road milage	Un-metalled road milage	Total road milage	Rate per cent sq. mile
II. To achieve the accessibility of : (i) 1 mile to a road in developed area; (ii) 3 miles in semi-developed area; (iii) 5 miles in undeveloped area.								
Milage required								
As per Crystal Pattern Formula	4,62,350	1,83,080	6,45,430	51.0	42,360	15,810	58,170	51.0

Note : This result indicates that with approximately the same target of road milages arrived at by Revised Grid and Star Formula, 1959, the accessibility can be reduced considerably to 1 mile from any road in developed area and 3 miles and 5 miles in semi and undeveloped areas respectively. In case of Uttar Pradesh, the target will still remain lower by 21 per cent and, therefore, these results indicate a glaring superiority of Crystal Pattern Formula over Revised Grid and Star Formula 1959.

	India				Uttar Pradesh			
III. To achieve the accessibility of (i) 1 mile from any road and 2 miles from a metalled road throughout								
Milage required								
As per Crystal Pattern Formula	5,83,785	2,57,180	8,40,965	66.4	52,250	23,000	75,250	66.4

Note : With an accessibility of 1 mile to a road the maximum road milage as per Crystal Pattern Formula comes to 66.4 miles per 100 sq. miles whereas according to Revised Grid and Star Formula, it is claimed that another 33 per cent increase in milage over 66 miles per 100 sq. miles will be necessary to achieve 1 mile accessibility.

Result : **The above inferences lead to the conclusion that it is proper to adopt Crystal Pattern Formula in preference to Revised Grid and Star Formula 1959.**

Appendix I

Relative Study of Road Statistics of Different Countries

Countries	Miles per 100 sq. miles of area	Miles per lakh of population
1. Great Britain	200.0	374
2. France	304.0	1,508
3. Belgium	232.8	310
4. Switzerland	183.1	593
5. Poland	173.1	790
6. Austria	166.5	815
7. West Germany	159.4	311
8. Hungary	109.9	406
9. U.S.A.	100.0	2,021
10. Italy	95.0	259
11. Czechoslavakia	88.0	354
12. India	25.0	82

Appendix I

Information to be Collected During Factual Survey under Different Sub-heads

(i) Area (a) Total area of a district;

(b) Area under cultivation;

(c) Cultivable area but not cultivated;

(d) Area under forest;

(e) Uncultivable area not available for cultivation and barren.

(ii) Population

(a) Dispersal and location of cities, towns and villages of graded population :

(i)	Below	500		—
(ii)	Between	500	to	1,000
(iii)	,,	1,001	to	2,000
(iv)	,,	2,001	to	5,000
(v)	,,	5,001	to	10,000
(vi)	,,	10,001	to	20,000
(vii)	,,	20,001	to	50,000
(viii)	,,	50,001	to	1,00,000
(ix)	,,	1,00,000	to	—

(b) Growth of population during last 20 years or so and its causes :

(i) By birth;

(ii) By migration and its causes;

(c) Possibilities of future growth.

(iii) Agricultural produce in country

(a) Crops grown, area sown for each crop and produce for different crops per acre;

(b) Export and import of agricultural produce, with origin and destination, quantity transported either way;

(c) Pedological conditions of soil with productivity in different areas.

(iv) Commercial & marketing centres

(a) Location of centres, their zones of influence, extent of transport to and from these centres for different commodities at different seasons of a year;

(b) Means of transport adopted.

(v) Industries

(a) Types of industries and their location ;

(b) Extent of import of raw materials with their sources ;

(c) Export of finished products, quantity, destination and directions ;

(d) Means of transport adopted ;

(e) Traffic other than transport for raw materials and products.

(vi) Miscellaneous

(a) places of fairs, pilgrimage, tourist centres and their locations ;

(b) Administrative headquarters, tehsil headquarters, police stations and outposts and their jurisdictions ;

(c) Post offices, hospitals, schools and their locations ;

(d) Existing means of communication system, its type and location—roads, rails, canals, rivers, bridged and unbridged crossings ;

(e) Topographical and geographical conditions of the countryside.

(vii) Engineering surveys

(a) Traffic census on various roads and census points all over ;

(b) Number and type of transport vehicles and their average movement in different directions ;

(c) Road use statistics ;

(d) Road life statistics ;

(e) Origin-destination surveys ;

(f) Period of maximum and minimum traffic ;

(g) Soil studies for procurement of road making materials ;

(h) Location of quarries for road materials.

Appendix III

List of Maps to Indicate the Factual Data Collected in Fact-Finding Survey of a District

(i) Map I – shall indicate :

(a) all the villages and towns of different population with different legends. Classification of area, viz., culturable under cultivation & unculturable.

(b) existing metalled and unmetalled roads, rails, rivers, canals and bridged and unbridged crossings, post offices, schools, hospitals, police stations, tehsil headquarters and parganas with their jurisdictions. places of fair, pilgrimage, industrial and commercial centres, markets and mandies.

(ii) Map II – shall indicate the varying intensities of productivity of the soil and its distribution in different areas, commercial, industrial and marketing centres and their zones and directions of import and export with probable intensities.

(iii) Map III – shall indicate the proposals of roads made by local people and authorities, the topography and geographical features of the country and the soil classification as mentioned in Map II above.

(iv) Map IV – shall indicate the latest traffic census along different roads, soil classification, commercial marketing centres & quarries for road materials.

Appendix IV

The following modified 'GRID AND STAR' FORMULAE formulae are proposed for milages of different classes of roads :

I. **National Highways**

$$\text{Milage} = \frac{A}{40} + \frac{B}{50} + \frac{C}{60} + \left(20K + 10 + \frac{M}{2} \right) + D\,;$$

II. **National Highways and State Highways**

$$\text{Milage} = \frac{A}{12.5} + \frac{B}{15} + \frac{C}{20} + (30K + 15M + 7N + P) + D\,;$$

III. **National Highways, State Highways and Major District Roads**

$$\text{Milage} = \frac{1}{5}\left(A + \frac{B}{2} + \frac{C}{3} \right) + (30K + 15M + 7N + 6P + 4Q + 1.5R) + D\,;$$

IV. **National Highways, State Highways, Major District Roads and Other District Roads**

$$\text{Milage} = \frac{3}{10}\left(A + \frac{B}{2} + \frac{C}{3} \right) + \left(30K + 15M + 7N + 6P + 8Q + 2.5R + \frac{5}{2} + \frac{T}{5} \right) + D\,;$$

V. **National Highways, State Highways, Major District Roads, Other District Roads and Classified Village Roads**

$$\text{Milage} = \frac{2}{5}\left(A + \frac{B}{2} + \frac{C}{3} \right) + \left(30K + 15M + 7N + 6P + 8Q + 3.7R + S + \frac{T}{2.5} + \frac{V}{8} \right) + D\,;$$

where A = Developed and agricultural area in square miles,

B = Semi-developed area in square miles,

C = Undeveloped and uncultivable area in square miles,

K = Number of towns with population over 1,00,000,

M = No. of towns with population between 1,00,000 and 50,000.
N = ,, ,, ,, ,, 50,000 and 20,000,
P = ,, ,, ,, ,, 20,000 and 10,000,
Q = ,, ,, ,, ,, 10,000 and 5,000,
R = ,, ,, ,, ,, 5,000 and 2,000,
S = ,, ,, ,, ,, 2,000 and 1,000,
T = ,, ,, ,, ,, 1,000 and 500,
V = ,, ,, ,, below 500, and
D = Allowance (assumed as 5 per cent) for future development and other unforeseen factors.

While calculating the milage of roads an allowance of 10 per cent increase in the milage in mountainous and hilly areas is to be made because of the circuitous nature of the alignments. In this connection the areas situated above 7,000 feet altitude need not be taken into account as such areas would need very few roads in view of sparse population.

Appendix V

Land usage, Distribution of Population, and milage Targets for India

(1) Distribution of area by land usage	Per 100 sq. miles	
(i) Agricultural & developed areas (A)	7,60,700 sq. miles	60.5 sq. miles say 3/5th
(ii) Semi-developed areas, forests, etc. (B)	2,59,600 sq. miles	20.5 sq. miles say 1/5th
(iii) Undeveloped & uncultivable areas (C)	2,46,600 sq. miles	19.0 sq. miles say 1/5th
	12,66,900 ,, ,,	100.0

(2) Number of towns and villages of different populations (1951 census)

Size of towns and villages (population)	Total No.	Symbol
Over 1,00,000	71	K
Between 50,000 & 1,00,000	93	M
,, 20,000 & 50,000	352	N
,, 10,000 & 20,000	652	P
,, 5,000 & 10,000	1,176	Q
,, 2,000 & 5,000	20,508	R
,, 1,000 & 2,000	51,769	S
,, 500 & 1,000	1,04,268	T
Below 500	3,80,019	V

Allowance for industries, future development and other specific aspects D (5 per cent)

Assumptions	Developed area	Semi-developed area	Undeveloped area
(i) Distribution of towns & villages with population of 2,000 and above	90 per cent	7 per cent	3 per cent
(ii) Distribution of towns & villages with population of less than 2,000	85 per cent	10 per cent	5 per cent

Appendix V-A

Targets for Uttar Pradesh

1. Area ... 1,13,409 sq. miles
2. Population ... 6,32,15,742
3. Area under cultivation ... 67,667 sq. miles (59.75 per cent)
4. Cultivable but not cultivated area. 16,681 sq. miles (15.00 ,,)
5 Area of uncultivable lands ... 29,061 sq. miles (25.25 ,,)
6. Number of towns and villages on the basis of 1951 census

Size of towns and villages (population)	Total No.	Symbol
Over 1,00,000	15	K
Between 50,000 & 1,00,000	14	M
,, 20,000 & 50,000	47	N
,, 10,000 & 20,000	74	P
,, 5,000 & 10,000	299	Q
,, 2,000 & 5,000	2,818	R
,, 1,000 & 2,000	10,272	S
,, 500 & 1,000	23,211	T
Below 500	75,458	V

1. INVESTIGATION OF SOME ROAD FAILURES IN NORTH WESTERN INDIA
2. BITUMINOUS PENETRATION MACADAM

"INVESTIGATION OF SOME ROAD FAILURES IN NORTH WESTERN INDIA"

By

DR. BH. SUBBARAJU, R.L. NANDA AND R. KRISHNAMACHARI*

CONTENTS

SYNOPSIS

Some experimental stretches of stabilised soil roads were constructed around Batala, Punjab (India), some years ago. These roads which performed well for a number of years were affected in some portions by floods in 1956.

Investigation of these experimental stretches was undertaken by the Central Road Research Institute, New Delhi, towards the end of 1956. Some of the roads constructed around Batala based on conventional methods were also covered by the investigation for a better understanding of the failures of some of the portions of these roads.

Various field tests were conducted on the affected and unaffected sections. The results of these tests and the conclusions drawn therefrom are presented in this Paper.

The affected stretches were reconstructed during January to May 1958 primarily consisting of Bituminous envelope for the road crust and designing the road crust using C.B.R. method of designing. So far the condition of the stretches is good.

* Central Road Research Institute, New Delhi.

1. INTRODUCTION

The country around Batala, a town in Gurdaspur District, Punjab, is of an alluvial deposit and, as such, is very flat in nature. The town with its surroundings was flooded by the waters of Kasur Nalla and Patti Nalla once in the year 1955 and twice in the year 1956. Even though the intensity of the flash flood which occurred in 1955 was far more higher than that of the flood in 1956, the duration of the flood in 1955 was only about three to four days whereas the flood in 1956 remained for weeks together followed by continuous heavy rain. Due to unusual high intensity of the rainfall and two consecutive floods, one towards the end of August and the other early in October 1956, the country around Batala was waterlogged unusually for a long time. In some of the portions of the country the water was stagnating on either side of the roadway for a period of more than six weeks. It is claimed by some that the duration of the floods in 1956 was the longest in the living memory.

Subsequent to the floods in 1956, some of the portions of the roads around Batala deteriorated rapidly.

After preliminary inspection the following roads were selected for intensive investigation:

1. Batala - Siri Hargobindpur Road, miles 2/5 to 18/5.
2. Batala - Qadian Road, miles 1/3 to 2/0.
3. Batala - Kahnuwan Road, miles 0/0 to 4/5.
4. Batala - Amritsar Road, 22nd mile.

Fig. 1 shows the general location of the four roads under investigation together with the other important roads around Batala. For detailed investigation, portions of roads both flooded and unflooded as well as affected and unaffected stretches in each category were chosen as representative lengths, in order to have a comparative study of the various sections.

As the field investigation was expected to be completed in about a fortnight due to the large number of tests, some of them time consuming, to be done at each selected location, only 13 locations were selected for detailed investigation.

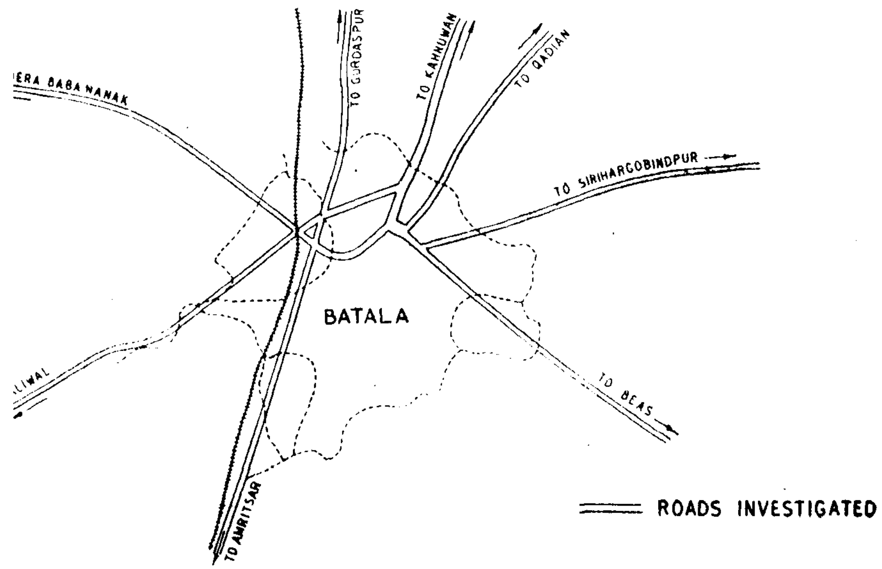

Fig. 1. A general plan of roads around Batala

2. FIELD INVESTIGATION

2.1. General

(a) Field tests conducted at each selected location

With a few exceptions, due to different types of road beds involved in the investigation, in general, the following field tests were conducted:

Field C.B.R. test:	On subgrade, base and wearing coat at the existing moisture content as well as after 24 hours soaking.
Plate Bearing test:	On subgrade, base and wearing coat at the existing moisture content as well as after soaking for 24 hours in a few cases.
North Dakota Cone Penetrometer Test:	On subgrade and base both at the existing moisture as well as after 24 hours soaking.
Field density Test:	On subgrade and base.
Moisture content:	On subgrade, base and wearing coat at the existing moisture content as well as after soaking. At different depths of subgrade from water table to the top of roadway.
Depth of water table:	At all selected locations.

Atterberg limits: Of subgrade soils in all cases; of base and wearing coat in some locations.

Mechanical analysis: Of subgrade soil; of base and wearing coat samples in some locations.

Sulphate content: Of subgrade and base soil samples.

In addition to the above mentioned tests on Batala-Siri Hargobindpur Road, levels were taken to a distance of about 200 ft on either side of the roadway at each of the test sections.

Note: 1. Existing moisture content — this refers to the moisture content existing in the road bed at the actual time of field test.

2. Wherever soaking was done, the test area was ponded with water for about 24 hours prior to the test.

(b) **Field testing technique**

For conducting the various tests on subgrade, base and wearing course, the plan adopted is as shown in Figs. 2 and 3.

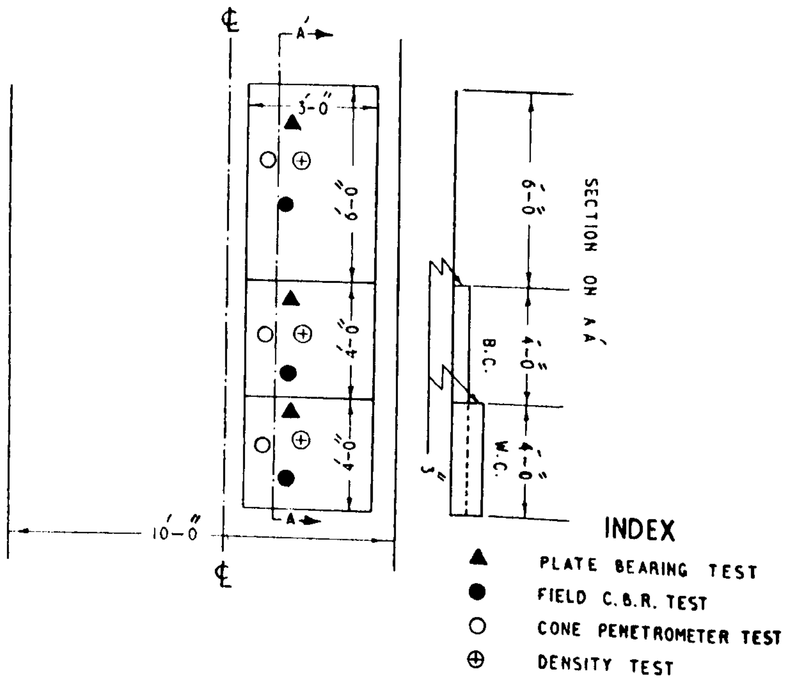

Fig. 2. General details of the test pit dug at each selected location

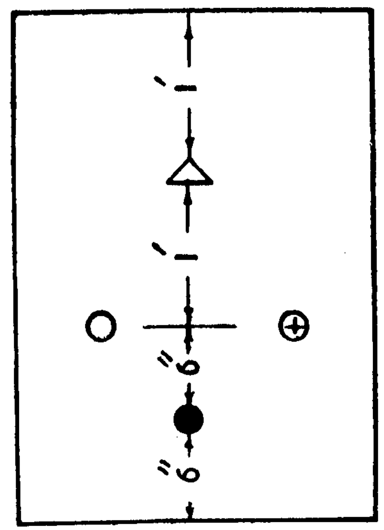

Fig. 3. Enlarged view of the disposition of various tests on a portion of a test section

Field C.B.R. test: The load was applied by means of a four-ton mechanical jack and was measured by means of a five-ton proving ring. The necessary reaction for the application of the load was obtained from a five-ton loaded truck. The rest of the procedure followed for the test was as per the general practice.

Plate Bearing test: The same loaded truck, jack and proving ring used for field C.B.R. test were used for the plate bearing tests also. Due to the limited reaction conveniently available in the field, the plate bearing tests were conducted only with 12 in. diameter plate. The plates used for the tests were of 12 in., 9 in., 6 in. and 4 in. diameters. To avoid eccentric loading, the load from the jack to the plate was transmittted through a ball and socket device. Two to three dial gauges were used to measure the settlement of the plate. The modulus of subgrade reaction presented in the data is calculated for a 12 in. diameter plate for 0.05 inch average settlement. The rest of the procedure followed was as per the general practice.

Moisture Content Determination: The moisture content determination for the various samples was done by alcohol method or by heating the sample containers by means of a pressure stove at low intensity.

Depth of Water Table: By digging a pit of about three feet by three feet in cross section on the berm, adjacent to the area of testing, the actual depth of ground water was measured in each case.

Field Density Test: The sand replacement method was used for measuring the field density.

For all the remaining tests, the standard procedures for the respective tests were adopted.

2.2. Investigation of Batala-Siri Hargobindpur road 2/5 to 18/5

2.2.1. Brief history

An experimental stretch of soil stabilised road was constructed on Batala-Siri Hargobindpur road at miles 2/5 to 18/5 during 1945. The road bed consisted of a stabilised base course of four and half inches loose compacted to three inches thick by means of sheeps foot rollers and a wearing coat, four and half inches loose, compacted to three inches thickness new by means of a smooth wheel roller. The wearing coat consisted of a mixure of two parts of graded soil and one part of brick ballast of one inch and downwards.

The road behaved very well till the disturbances started due to partition of country. As there was no maintenance during the period of disturbances and as extremely heavy traffic passed over the first eight and half miles due to the exchange of population, the wearing course had completely worn out and the traffic was using the base course. In due course the base course also was damaged.

Hence the road bed was reconstructed during 1950-1951 with similiar base and wearing coat together with graft stone of 1½ in. size in the wearing course at the rate of 6 cubic feet per

100 square feet and a surface treatment using 6 cubic feet of chips per 100 square feet.

The performance of the road was quite satisfactory till about October 1956. But soon after the second flood of 1956 in October, some of the stretches in mile 2/5 to 11/5 were found damaged.

2.2.2. Field test portion

With a view to make a comparative study of the affected and unaffected stretches, eight portions were selected for study and the details of the same are furnished in Table 1.

TABLE 1

Details of the test locations on Batala-Siri Hargobindpur Road

Mile/Furlong	Description
6/6	Flooded and unaffected.
6/7	Flooded and affected by extensive patches combined with rutting.
7/8	Flooded and affected by extensive patches combined with rutting.
8/2	Flooded and unaffected.
8/4	Not flooded but affected in patches.
8/7	Not flooded and unaffected.
11/4	Not flooded and unaffected so far. But symptoms of cracking are seen on scattered areas.
11/5	Not flooded but affected, cracking and ravelling of surface.

Photo 1 shows the stepped up test pit dug at mile 8/2 before test and Photo 2 shows the test pit under soaking in mile 8/7. Photo 3 shows the field laboratory. The traffic intensity at various points during the past seven years is shown in Table 2.

TABLE 2

Traffic census on Batala - Siri Hargobindpur road for the period 1949 to 1956

Period	Mile 2/5		Mile 3/2		Mile 4/3		Mile 13/2		Remarks
	Motor vehicle traffic in tons	Non-motor vehicle traffic in tons	Motor vehicle traffic in tons	Non-motor vehicle traffic in tons	Motor vehicle traffic in tons	Non-motor vehicle traffic in tons	Motor vehicle traffic in tons	Non-motor vehicle traffic in tons	
1949	—	—	—	—	—	—	182	443	
June 1952	—	—	—	—	89	76	—	—	
Dec. 1952	—	—	182	65	257	227	—	—	
1953-1954	—	—	188	108	—	—	—	—	
1954-1955	—	—	105	122	—	—	—	—	
1955-1956 (1st half)	877	400	—	—	—	—	—	—	Traffic was diverted to this road due to repairs on a near-by road. Hence there was sudden increase.
1955-1956 (2nd half)	832	143	—	—	—	—	—	—	

Note: (i) The motor vehicles covered in this Table are of 3 to 5 tons.

(ii) The traffic intensity given is in tons for 24 hours.

The results of the various tests are presented in Table 3 and Figs. 4 to 7.

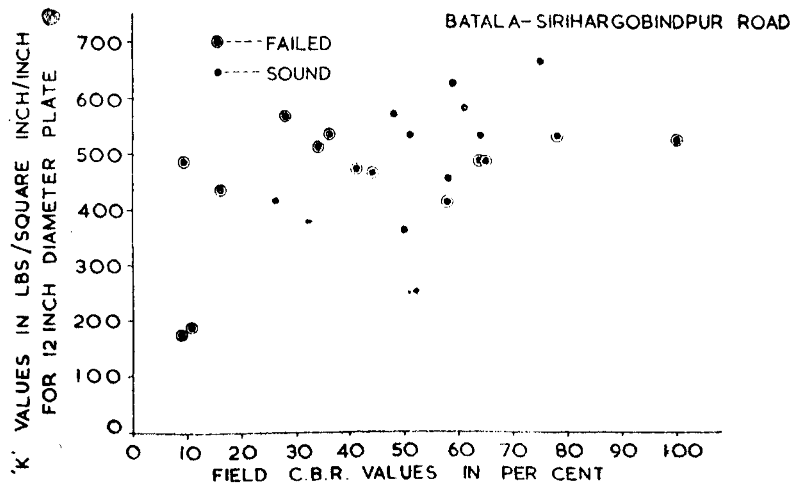

Fig. 4. **Correlation between C. B. R. values and K. values**

2.2.3. **Discussion of Test Results**

Before going into the test data in detail, it is worth while taking stock of few external factors contributing to the failures in patches on Batala-Siri Hargobindpur road.

(i) **Water-logged area**: The fly levels together with the cross sections taken for a distance of two miles on the distressed section of the road indicate that the road runs almost at the adjoining ground level without any embankment.

Even after one and a half months after the floods had receded, the road was still water-logged .in some portions. Photo 4 shows the water-logged area in mile 7/8, about 6 weeks after the flood had receded.

(ii) **Rise in ground watertable**: The data collected by the Public Works Department, Punjab, indicate that there is a rise in water table anywhere from zero to fifteen feet in this locality in the course of last seven years. Batala-Siri Hargobindpur road runs almost in between the two grids of the observation wells. Fig. 8 shows the location of the observation wells, and Table 4 presents the rise in water table at the various observation wells.

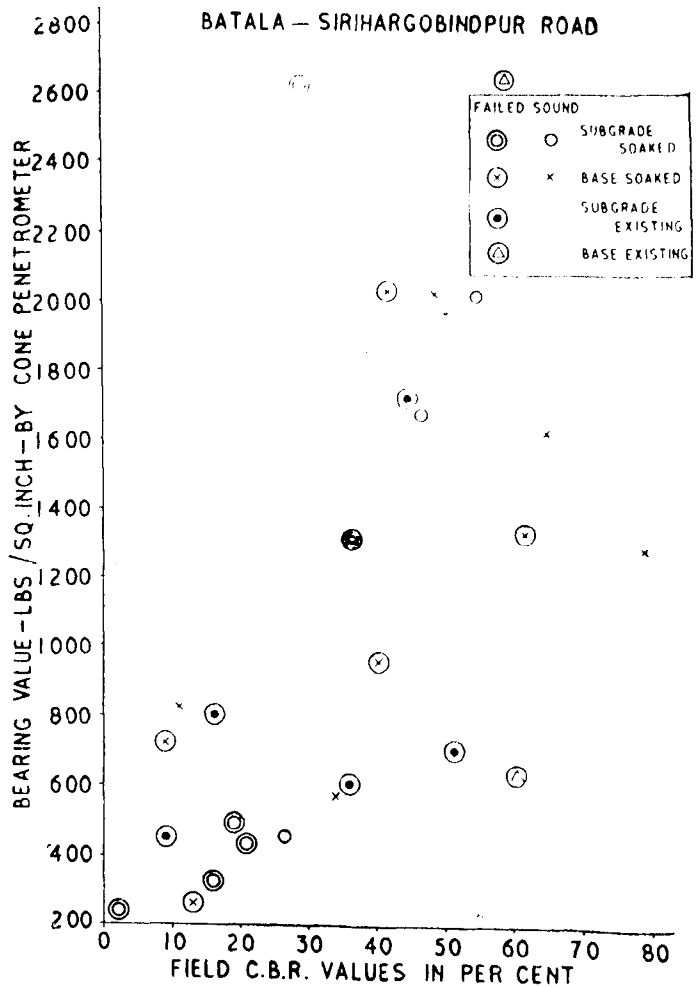

Fig. 5. **Correlation between C. B. R. values**

It is clear from the results in Fig. 6 that the depth of water table is only at two to four feet below the road surface.

(iii) **Effect of floods**: There were two floods in quick succession in 1956. The duration of the floods in 1956 had been far longer than that of 1955.

(iv) **Continuity of increased traffic during and after floods:** The important point that is to be remembered is the continuity of

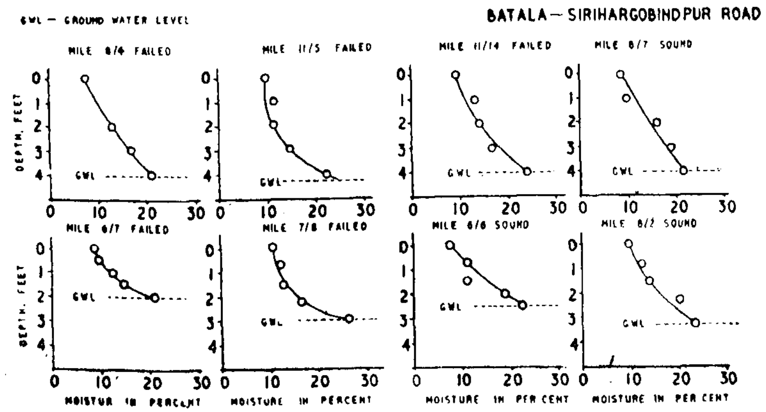

Fig. 6. **Moisture variation from the ground water level to riding surface on B. S. road**

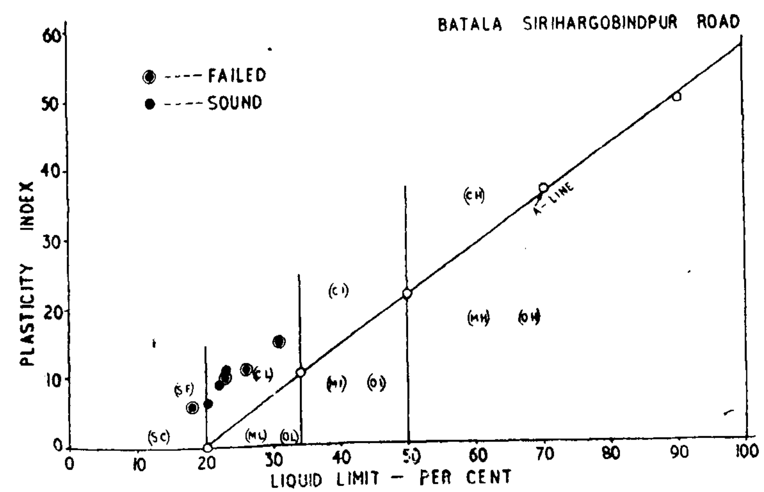

Fig. 7. **Plasticity chart for subgrade soils**

the traffic during and after floods. The close proximity of the ground water table aided by the side stagnated water had softened the subgrade considerably. The continuous and increased traffic on the highway with this softened subgrade was one of the potential causes for resulting in sinking of the subgrade and its after effects.

TABLE 3

Results of strength tests on Batala-Siri Hargobindpur road

Mile/Furlong	Existing condition of test area	C. B. R. Test—Per cent						Plate Bearing Test—12" Dia. Plate—'K'—values in psi./in. for 0.05 in. settlement						Bearing value—psi. by cone penetrometer test			
		Wearing course		Base course		Subgrade		Wearing course		Base course		Subgrade		Base course		Subgrade	
		Existing moisture	After soaking	Existing moisture	After soaking	Existing moisture	After soaking	Existing moisture	After soaking	Existing moisture	After soaking	Existing moisture	After soaking	Existing moisture	After soaking	Existing moisture	After soaking
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
6/7	Failed	65	—	64	61	36	16	492	—	492 } 521 }	—	538	—	1600	1320	600	320
7/8	Failed	100	—	78	40	44	21	526	—	538	—	470	—	1275	950	1700	430
8/4	Failed	—	60	—	41	—	28	—	—	—	475	—	571	—	2000	—	2600
11/4	Failed	58	—	34	13	9	2	418	—	515	—	486	—	570	260	450	240
11/5	Failed	9	—	11 } 7 }	9	16	19	175	—	181	—	436	—	820	720	800	490
6/6	Sound	77 } 64 }	—	61	60 } 65 }	51	26.5	538	—	589	—	*215 †538 }	—	3000	630	700	450
8/2	Sound	75	—	48	36	62 } 50 }	46	673	—	575	—	368	—	2000	1300	3000	1650
8/7	Sound	—	26	—	58	—	59	—	418	—	464	—	633	—	2600	—	1990

Note: * Clay layer. † Sand layer.

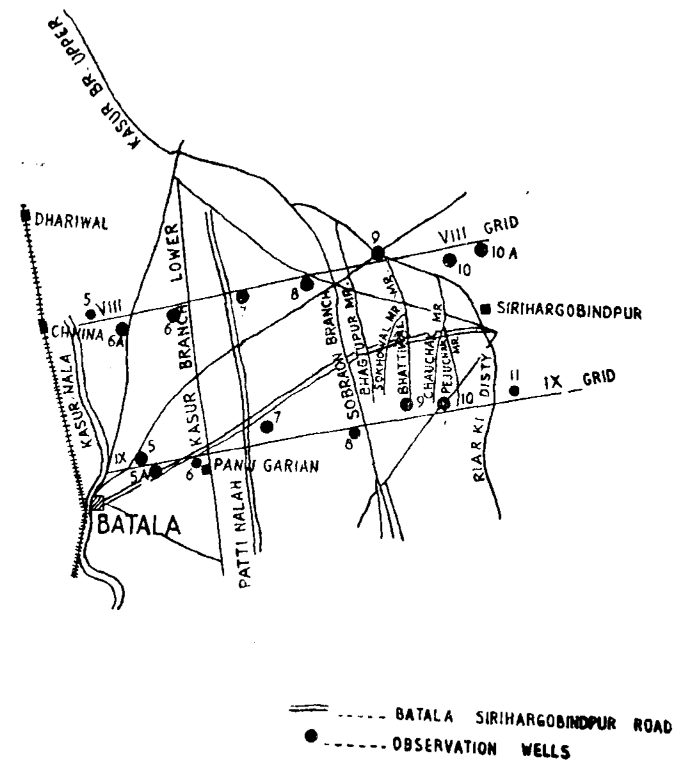

Fig. 8. Plan showing the observation wells for measuring ground water level on either side of Batala-Siri Hargobindpur road

By the time this investigation was taken up, there was no surface water in the adjoining fields except at a few places and in road dam portions. This shows the stage of ingress of the moisture wave from the subgrade. With a view to simulate the moisture condition at the time of floods, the field testing was done before and after soaking. Even though the test pits were soaked for 24 hours for the tests conducted after soaking, depending upon the type of soil, the degree of saturation due to such soaking is of course questionable compared to the soaking done for the laboratory samples or saturation of subgrade as a result of rise in water table.

Results in Table 3 indicate that the field C.B.R. values of all failed sections before soaking range from 9 to 44 whereas they reduce to 2 to 28 after soaking.

TABLE 4

Rise in ground water table at the observation wells near Batala - Siri Hargobindpur road

Grid VIII

	Well Nos. in the Grid						
	6	6A	7	8	9	10	10A
Rise in feet between 1950 and 1956	3.2	Nil	5.7	10.6	7.5	3.3	0.5

Grid IX

	Well Nos. in the Grid							
	5	5A	6	7	8	9	10	11
Rise in feet between 1950 and 1956	5.6	4.6	3.6	16.2	18.2	15.5	9.4	0.3

In the case of sound portions, the values of field C.B.R., after soaking, vary from 27 to 59. This indicates two important features :

1. The non-uniformity of the subgrade soil.
2. The effect of prolonged flood on the subgrade soil.

The low sand content, the high percentage of clay and silty material and the high plasticity index of subgrade soil in the failed sections are all contributing factors for a weak subgrade. These characteristics contribute to the greater absorption of moisture and thus resulting in poor bearing power.

The longer duration of flood in 1956 and that too, twice in a short interval of time had contributed to the subgrade absorbing greater amount of moisture than what it had absorbed during the short duration of the flood in 1955. This might be the reason for the non-failure of some of the failed portions, during the flood in 1955, and failing during the floods in 1956.

From the traffic census given in Table 2, it will be seen that the motor vehicle traffic intensity rose from 165 tons in 1952 to about 1000 tons in 1955/56 per day. Based on the classification of Road Research Laboratory, Harmondsworth, the traffic as stated above on Batala-Siri Hargobindpur Road is classified as class C for interpretation of the C.B.R. test results.

The curves developed by the Road Research Laboratory, Harmondsworth, U.K., are based on the laboratory C.B.R. values and are given in Fig. 9. Hence for proper interpretation of the field C.B.R. values obtained in this investigation, it is essential to derive the relationship between laboratory and field C.B.R. values. Even though it is possible in the laboratory to compact the specimen to the same density and moisture content as existed in the field, it is not possible to reproduce the same soil structure as that existing in the field. Research so far done on this aspect is very limited. Some of the studies made * indicate that, for the particular type of soils and traffic involved in the studies, the values of field C.B.R. are any where from one to two times the values of the laboratory C.B.R., depending upon the range of values.

Limited laboratory C.B.R. tests had been done on some subgrade soils covered in this investigation. For conducting the laboratory C.B.R. test, it would have been better if the samples were cut from the road itself with a C.B.R. mould and a cutting collar. But this procedure could not be adopted in the field for want of proper equipment for taking undisturbed samples in the field. The results in Table 5 show the comparative values of field C.B.R. after soaking for 24 hours in situ and the laboratory and C.B.R. values with 4 days soaking. Even though it was realized that greater number of samples for laboratory C.B.R. test would have given more reliable data, the limited data presented indicates a regular trend and consistency. The moisture content in the field of the three subgrade samples after soaking varied from 10.6 to 13.4 per cent whereas, in the laboratory, the same varied from 12.6 to 18.6 per cent. This clearly indicates that the 24 hours soaking effect in the field is not simulating the laboratory conditions.

Hence in order to evaluate the subgrade strength using C.B.R. curves evolved by the Road Research Laboratory (Harmondsworth) it is more appropriate to take laboratory C.B.R. values than the field C.B.R. values after soaking. From

* "Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavements." By Baker, R. F. & Drake, W.B.; Engineering Experiment Station Bulletin No. 13., Sept. 1949, University of Kentucky, U.S.A., Page 42.

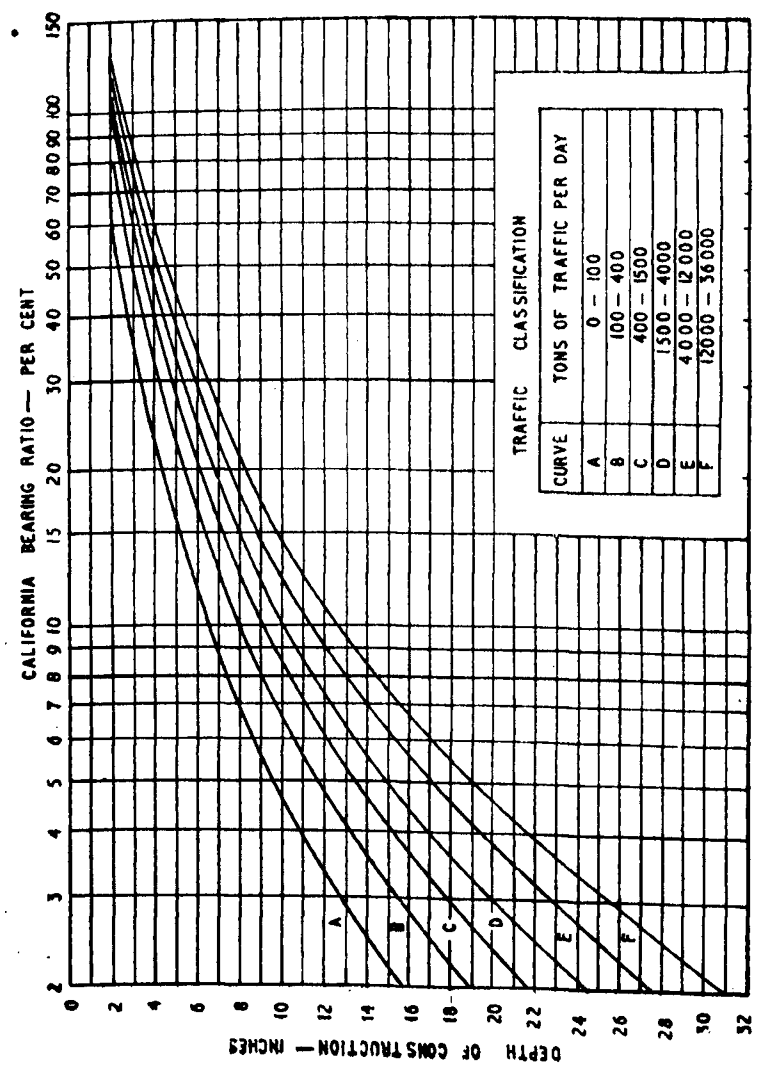

Fig. 9. Proposed C. B. R. design curves for different classes of road.

Photo 1.

Stepped up pit before soaking at mile 8/2 of Batala-Siri Hargobindpur road

Photo 2.

Stepped up pit under soaking for 24 hours at mile 8/7 of Batala-Siri Hargobindpur road

Photo 3.
The field laboratory of the Central Road Research Institute at Batala

Photo 4.
General view of the waterlogged area on both sides at mile 7/8 of Batala-Siri Hargobindpur road

Photo 5.
Cross-section of mile 7/8 (failed section) of Batala-Siri Hargobindpur road. The uniform thickness of wearing course and the complete sinking of all the layers may be seen

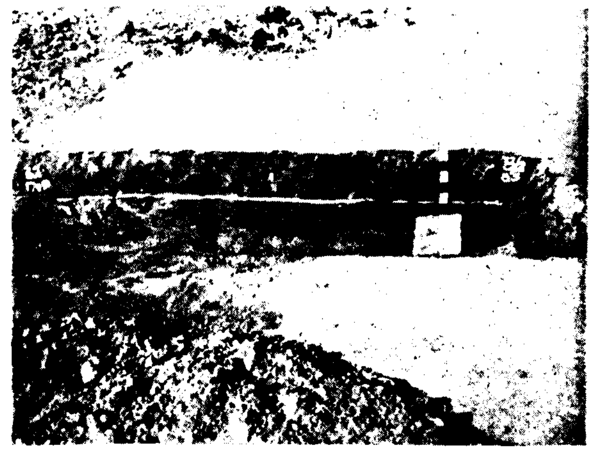

Photo 6.
Cross-section of mile 8/2 (sound section) of Batala-Siri Hargobind-pur road

Photo 7.
Close up view of bituminous surfacing of mile 11/4 of Batala-Siri Hargobindpur road showing the numerous cracks and surface ravelling

Photo 8.
The ravelled bituminous surface at mile 11/5 of Batala-Siri Hargobindpur road. The lighter traffic prefer to use the berms

Photo 9.
The bituminous surface at mile 8/2 of Batala-Siri Hargobindpur road showing the absence of any rutting

Photo 10.
The crack pattern and ravelling on Amritsar-Batala road after being affected by floods

Photo 11.
The two wheel tracks formed on Batala-Kahnuwan road mile 3/5

Photo 12.
General view of reconstructed portion (mile 7/4) of Batala-Siri Hargobindpur road after a year's traffic

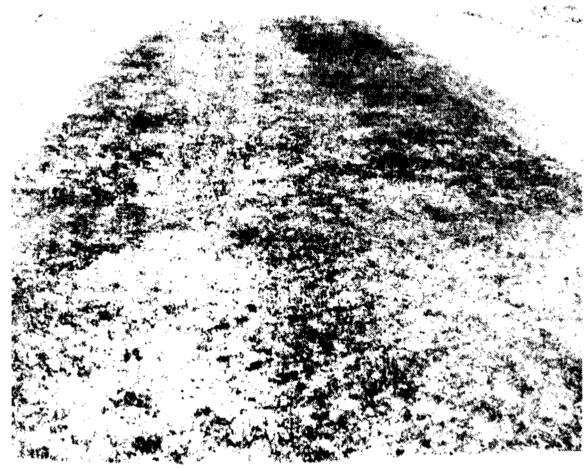

Photo 13.
Close up view of the surface of the reconditioned portion of 7/4 of Batala-Siri Hargobindpur road after 1 year of traffic

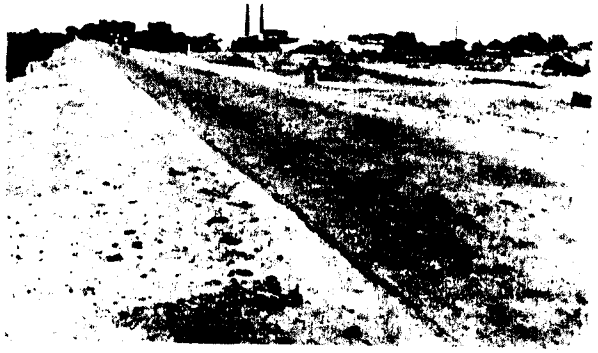

Photo 14.

Reconstructed portion (mile 8/3) of Batala-Siri Hargobindpur road after 1 year of traffic

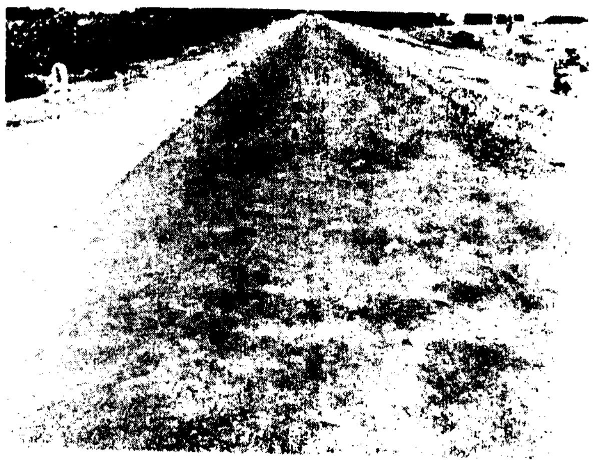

Photo 15.

Close up of the surface of the re-conditioned portion of 8/3 of Batala-Siri Hargobindpur road after 1 year of traffic

TABLE 5

Results of C.B.R. values of a few failed subgrades of Batala-Siri Hargobindpur road

Location Mile	Field moisture per cent	Field density	LAB. C.B.R. Per cent — Specimen compacted to field moisture and density. Period of soaking — 4 days — Trial I — C.B.R.	Trial I — Swelling per cent	Trial I — Moisture per cent	Trial II — C.B.R.	Trial II — Swelling per cent	Trial II — Moisture per cent	15 days — C.B.R.	15 days — Swelling per cent	15 days — Moisture per cent	Field C.B.R. Per cent — At existing moisture — C.B.R.	At existing moisture — Moisture per cent	After 24 hours soaking — C.B.R.	After 24 hours soaking — Moisture per cent	LAB. C.B.R. — Specimen compacted at optimum moisture and Maximum density — 4 days soaking — Opt. moisture	Density	C.B.R.	Swelling per cent	Moisture per cent
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
7/8	9.3	125.9 lb/cu.ft. (2.01) gm/cc	4.8	1.90	12.57	5.30	1.88	13.5	3.70	1.87	14.04	44	9.3	21	10.6	10.2	126.5 lb/cu.ft. (2.03) gm/cc	6.0	0.44	11.51
11/4	10.6	101.0 lb/cu.ft. (1.62) gm/cc	Nil	0.32	18.41	0.43	0.36	17.8	0.12	0.40	18.34	9	10.6	2	13.4	9.4	127.0 lb/cu.ft. (2.03) gm/cc	6.4	0.44	10.70
11/5	10.2	116.3 lb/cu.ft. (1.86) gm/cc	2.0	5.2	18.61	2.40	3.46	18.8	2.05	3.58	18.94	16	10.2	19	13.4	11.8	122.4 lb/cu.ft. (1.96) gm/cc	9.6	0.86	15.03

the values of the laboratory C.B.R. for the three subgrade samples covered in Table 5, the required superimposed total thickness of the road bed for satisfactory performance under the prevailing traffic works out to about 13½ inches, whereas the existing thickness of the pavement laid at these locations was 6½ inches. Judging from C.B.R. method of design, this indicates that the pavement failure at these three places is due to the settlement of subgrade. Photo 5 showing the section of the road bed under the wheel track in mile 7/8 clearly indicates the bending of base and wearing coat, resulting due to the sinking of the subgrade. In comparison, Photo 6 shows the sound portion at mile 8/2 where the uniform thickness of wearing course and base with the right camber is retained.

The significant fall in strength on account of ingress of moisture in the sub-grade as shown by the field C.B.R. values is also confirmed by the results of North Dakota Cone Penetrometer test. Table 3 shows that the bearing power of the subgrade on soaking, varies from 240 to 490 pounds per square inch in the case of failed sections and from 450 to 1990 in the case of sound sections. At existing moisture condition, the subgrade has given a bearing power varying from 450 to 1700 pounds per sq. in. for the failed sections and 700 to 3000 pounds per sq. in. for the sound sections.

A study of Table 3 regarding the strength of base course at existing moisture and soaked conditions, both in the failed and sound stretches, would reveal that there is no significant fall in the strength by the ingress of moisture except in stray cases. In the case of base course, there is no significant change in field C.B.R. values before and after soaking, as compared to subgrade. The field C.B.R. value of base course after soaking varies from 40 to 60. As already stated in para 2.2.3, the variation between laboratory and field C.B.R. values is of the order of one to two, the variation getting decreased with increase in C.B.R. values. Even taking the very conservative variation of 2, the laboratory C.B.R. value of base after soaking, for an average field C.B.R. value of 50, works out to 25. From the C.B.R. curves the thickness required for the prevailing traffic and for a laboratory C.B.R. of 25 is 5½ inches, whereas the existing thickness of wearing course and surfacing varies from 4½ to 5 inches. Hence this again indicates that it is a subgrade failure.

The crack pattern on the bituminous surfacing occurring in a few cases could be understood properly if the mechanism of failure is dealt with in detail. Under the waterlogged condition, the strength of the subgrade has been considerably decreased as evidenced by the C.B.R. values, after soaking. Consequently the subgrade with the insufficient thickness of the superimposed road bed sank under passing traffic. As a result of the sinking, high tensile stresses developed on the top bituminous surface, thus cracking the same.

In such cases, if water comes on top, infiltration takes place from top to bottom and thus further deteriorating the subgrade and intensifying the cracking. The asphaltic surfaces taken at miles 11/4 and 11/5 and believed to be cracked on the above mentioned analogy are shown in Photos 7 & 8.

With the cracking up of the asphaltic surface, ravelling of the surface takes place by the passing traffic as will be seen from the photograph taken at mile 11/5, Photo 8. The traffic prefers to use the berms than the ravelled road.

Continued motor traffic over this ravelled section disturbs completely the wearing course and disturbs the base as well. It is quite probable that the variation in the strength of base course at some failed sections on soaking is due to the disturbance by traffic as described above.

Observations at miles 6/7, 7/8 and 8/4 show that the deep rutting and destruction of surfacing and wearing course occurred, by the continued use of the affected sections by mixed traffic. In comparison, the photograph of the sound section taken at mile 8/2 is shown in Photo 9.

The consistency in field C.B.R. and cone penetrometer values at mile 8/4 both for base and subgrade, even after soaking indicates that the depression caused at the area of the patch may be due to surface failure. The subgrade soil at this test location has a sand content as high as 46 per cent and a plasticity index of 9.9 per cent. These characteristics, coupled with high strength even after soaking, show that the subgrade is strong enough but the failure may be caused from top. As the tar surfacing was laid about five and half years back and in the absence of any further renewal coat, the surfacing has shown signs of excessive wearing and oxidation of tar. It is quite probable that the failure at this test location and some scattered portions in this furlong may be due to the worn out surfacing. With the gradual wearing of the surface, the traffic might have used the wearing course, causing rutting.

The above assumption is confirmed by an examination of the cross section of the road bed at the test location, which revealed the absence of any depression of the interface of the base course and subgrade.

The poor soil characteristics, the significant fall in strength on ingress of moisture and the inadequate thickness of road bed for the traffic encountered, all contribute to the sinking of the subgrade as noticed in the failed sections at miles 7/8, 11/4 and 11/5.

Thus the results of this investigation indicate that the main cause for failure at miles 6/7, 7/8, 11/4, and 11/5 is due to softening of the subgrade, whereas at mile 8/4 the failure is due to infiltration of water from top through the cracked bituminous surfacing.

2.3. Investigation of other roads around Batala

2.3.1. Brief History

A Batala-Kahnuwan Road mile 0/0 to 4/5

An experimental stretch of soil stablised road was constructed on Batala-Kahnuwan Road in May 1955. The road bed consisted of a stabilised soil base course of four and half inches loose compacted to three inches thick by means of sheepsfoot rollers and a wearing coat of four and half inches loose compacted to three inches thick by means of a smooth wheel roller. The wearing course consisted of a mixture of two parts of graded soil and one part of brick ballast of one inch and downwards. The wearing course was surface treated with 33 lb of tar and three and three-fourths cubic feet of $\frac{3}{4}-1/8$ in. aggregate per 100 sq. ft.

A second coat of surface treatment was given in May 1956 with 22 pounds of bitumen and two and quarter cubic feet of $3/8-1/8$ in. aggregate per 100 sq. ft.

In some of the low-lying areas, deep wheel tracks had been formed.

B. Batala-Qadian Road mile 0/0 to 2/0

This road was constructed in February 1955 with the conventional tye of construction. The soling consisted of one flat brick with the frog down on the existing soil subgrade and the base consisted of three inch loose water bound macadam compacted to two and half inches thick. The water bound macadam was surface dressed with 45 pounds of tar and four and half cubic feet of $\frac{3}{4}$ to 1/8 in. aggregate per 100 sq. ft. The road was again surface dressed in May 1956 with 22 pounds of bitumen and three cubic feet of 3/8 to 1/8 in. aggregate per 100 sq. ft.

The road was not affected by the floods.

C. Amritsar-Batala Road mile 22/2

This road is the Grand Trunk Road connecting Amritsar and Pathankot *via* Batala. The road is of conventional type of construction using brick soling, water bound macadam and surface dressing. The surface dressing was renewed from time to time.

Some portions of the road were considerably affected by the recent floods. The surface cracked and ravelled extensively.

2.3.2. Field test locations

The details of field test locations selected for study are given in Table 6.

The traffic intensity of Batala-Kahnuwan Road and Batala-Qadian Road is given in Table 7.

The results of various tests are presented in Tables 8 to 11.

TABLE 6

Details of the test locations

	Mile/Furlong	Description
A. Batala-Kahnuwan Road		
	0/7	Unflooded and Unaffected.
	1/5	Flooded – Cracking and slight sinking are seen.
	3/5	Flooded and affected.
B. Batala-Qadian Road		
	1/6	Flooded and unaffected.
C. Amritsar-Batala Road		
	22/2	Flooded and affected.

TABLE 7

Traffic census

A. Batala-Kahnuwan Road		
	Date of census	– 24-5-1956.
	Motor traffc	– 16.5 tons for 24 hours.
	Non motor traffic	– 24.0 tons for 24 hours.
B. Batala-Qadian Road		
	Date of census	– 27-5-1956.
	Motor traffic	– 83.5 tons for 24 hours.
	Non Motor traffic	– 46.5 tons for 24 hours.

TABLE 8

Results of strength tests

Mile Furlong	Existing Condition of test area	C. B. R. Test — (Maximum) Per cent						Plate Bearing Test – 12 in. Dia. plate—'K' values in P.S.I. inch for 0.05 settlement						Bearing Value – P.S.I. by cone penetrometer test			
		Wearing course		Base course		Subgrade		Wearing course		Base course		Subgrade		Base Course		Subgrade	
		Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking	Existing moisture	After 24 hours soaking
A. Batala-Kahnuwan Road																	
0.7	Sound	—	—	53	—	38	—	—	—	—	—	382	—	2000	—	400	—
1.5	Failed	—	—	47	19	38	30	—	—	—	—	339	—	630	520	760	445
3.5	Failed	—	—	38	29	41	38	—	—	—	—	464	—	750	380	1400	420
B. Batala-Qadian Road																	
1/6	Sound	—	—	—	—	—	23	—	—	—	—	—	566	—	—	—	420
C. Amritsar-Batala Road																	
22/2	Failed	—	—	—	—	20	15	—	—	—	—	—	—	—	—	1400	620

TABLE 9

Field density and moisture content at the test location

Mile/Furlong	Existing condition of test area	Dry Bulk Density in grams per c.c.		Moisture — Per cent						Depth of water table from berm surface in inches
		Base course	Subgrade	Wearing course		Base course		Subgrade		
				Existing	After 24 hours soaking	Existing	After 24 hours soaking	Existing	After 24 hours soaking	
A. Batala-Kahnuwan Road										
0/7	Sound	1.7	1.60	8.1	—	10.0	—	12.1	—	63
1/5	Failed	1.8	1.60	8.4	—	8.8	10.9	10.3	12.1	18
3/5	Failed	1.7	1.70	9.0	—	10.6	12.0	12.1	13.0	30
B. Batala - Qadian Road										
1/6	Sound		1.53	—	—	—	—	11.8	12.5	24
C. Amritsar - Batala Road										
22/2	Failed	—	1.80	—	—	—	—	10.3	12.9	24

TABLE 10

Results of atterberg limits, sand content and sulphate content of soil-aggregate mixtures in the wearing course and Soil-sand mixtures of base course

A. Batala - Kahnuwan Road

Mile Furlong	Existing condition of test area	Atterberg Limits			Sand content per cent	Sulphate content per cent	Remarks
		Liquid limit	Plastic limit	Plasticity index			
		Base Course					
0/7	Sound	19.50	13.05	6.45	56.0	Nil	
		Base Course					
1/5	Failed	19.68	12.67	7.01	58.6	Nil	
		Wearing Course					
3/5	Failed	21.50	10.77	10.73	34.5	Nil	
		Base Course					
		19.83	14.74	5.09	49.5	Nil	Sand not well mixed separate sand portions were observed.

TABLE 11

Results of atterberg limits, sand and sulphate contents of subgrade soil on

Mile/ Furlong	Existing condition of test area	Atterberg limits			Sand content per cent	Sulphate content per cent	Remarks
		Liquid limit	Plastic limit	Plasticity index			
	A. **Batala - Kahnuwan Road**						
0/7	Sound	32.84	15.01	17.83	14.9	Nil	
1/7	Failed	20.40	13.73	6.67	56.0	Nil	
3/5	Failed	25.34	15.09	10.25	32.0	Nil	
B. Batala - Qadian Road							
1/6	Sound	25.20	13.87	11.33	45.0	Nil	Black in colour
C. Amritsar Batala Road							
22/2	Failed	28.24	14.67	13.57	28.5	Nil	

2.3.3. **Discussion of Test Results**

Investigation of Amritsar-Batala Road and Batala-Qadian Road were undertken with a view to have a comparative study of affected and unaffected sections both in the conventional and soil stabilized road constructions.

Table 8 shows that the field C.B.R. values of the subgrade do not indicate any appreciable fall after soaking.

From the field C.B.R. values of subgrade on soaking, and for the traffic intensity of Batala-Qadian Road as in Table 7, the thickness of road bed is quite adequate.

The Amritsar-Batala Road is a state highway with a traffic intensity of about 2000 tons per day. The field C.B.R. of subgrade after soaking was 15 only. From the general trend of differences between field C.B.R. values and comparable laboratory C.B.R. values, taking the laboratory C.B.R. as 7, for the above mentioned traffic on this road, the thickness required for successful performance works out to 12 inches. But the existing thickness of the road at the time of investigation was only about 8 inches. This indicated that the failure of the road was due to inadequate thickness of the road crust for the prevailing subgrade and traffic. The crack pattern and ravelling on Amritsar-Batala Road after being affected by floods, is shown in Photo 10.

In miles 1/5 and 3/5 of Batala-Kahnuwan Road, the field C.B.R. values of base show an appreciable fall on soaking, on account of comparatively low density. The slight settlement in the wearing course has led to surface cracking and slight wheel tracking in mile 1/5 of Batala-Kahnuwan Road. But at mile 3/5 the settlement of the wearing course was appreciable and it has slowly developed into the base course as well, resulting in deep wheel tracks. An examination of cross section of the road revealed that the thickness of the wearing course is reduced at the rutted portion in comparison to the thickness of the wearing course in the adjacent side. The wheel tracks of this furlong are shown in Photo 11. With only bullock-cart traffic on this road and with the resulting high intensity of stress on top and with the decreased depth of stress influence, it seems, with the low C.B.R. values after soaking, the base and wearing course got consolidated under the wheels resulting in wheel tracks.

In mile 0/7 of Batala-Kahnuwan Road, the road runs on a high embankment. The Plasticity index of subgrade is as high as 18 per cent and the sand content of the same is as low as 15 per cent. This furlong had not shown any signs of deterioration.

This might be due to the presence of high embankment resulting in creating less detrimental effects due to nonflooding of the roadway. The depth of water table at this furlong is more than five feet.

3. CONCLUSIONS

(1) The failure in patches on Batala-Siri Hargobindpur Road in miles 6/7, 7/8, 11/4 and 11/5 is due to the *sinking of the subgrade caused by ingress of excess moisture.*

(2) The failure on Batala-Siri Hargobindpur Road as in mile 8/4 might be due to the *surface failure.*

(3) The failure in patches on Batala-Kahnuwan Road might be due to the *settlement in the wearing course* and *base itself.*

(4) The failure of Amritsar-Batala Road might be due to *inadequate thickness of pavement* over the subgrade with its decreased bearing value due to the ingress of excess moisture.

(5) The cause for the successful performance of the Qadian Road, with conventional method of construction, seems due to the fact that the existing thickness is adequate from the C.B.R. design point of view.

4. RECOMMENDATIONS

(1) In order to minimise the effects of capillarity occurring due to high water-table and possible floods, it is recommended to adopt one or more of the following alternatives using the locally available material. The particular method to be adopted should be arrived at on examining the cost analysis of the various alternatives.

(a) Running the road on embankment. The height of the embankment to be constructed from locally available soils should be kept at the required minimum from the point of the highest level of water in the adjoining field.

(b) Stabilization either with cement or lime.

(c) Enveloping the road crust completely with an impervious layer.

5. RECONDITIONING THE FAILED SECTIONS

The damaged portions 6/7 to 9/0 of Batala-Siri Hargobindpur Road were reconditioned during January to May 1958, adopting the following specifications :

(a) The existing crust was dug up and the subgrade below was levelled and rolled.

(b) Shell primer No. 1 (50 per cent of bitumen 80/100 and 50 per cent diesel oil) was spread over the prepared subgrade including on sides of the road crust at the rate of 40 lb per 100 sq. ft. in two coats and allowed to soak through each time.

(c) Immediately above the shell primer, a 9 in. loose thickness of sulphate free natural soil was spread and compacted to 6 in. thickness at optimum moisture with a 6 ton power roller.

(d) Over the compacted natural soil a base course and a wearing course each of 3 in. thick compacted thickness were constructed together with grafting following *Mehra's method of stabilization.*

(e) Surface treatment consisting of two coats of Shalimar Road Tar No. 3 using 60 lb per 100 square feet (for both the coats) and 3.5 cu. ft. per 100 sq. ft. of ½ in. chips in the 1st coat and 2.5 cu.ft. per 100 sq. ft. of ¼ in. chips in the second was laid over the grafted wearing course.

(f) Brick on edge was provided on both sides.

The road is giving satisfactory performance so far and photos taken in June 1959, one year after reconstruction, are presented in photos 12 to 15.

6. SUGGESTIONS FOR FURTHER RESEARCH

I. The correlation between field and laboratory C.B.R. values should be established for proper interpretation of C.B.R. design curves using field C.B.R. values.

II. Study of available literature on the effect of variation in rate of loading for the C.B.R. test should be conducted. Then, if necessary, the study to have a proper understanding of the variation in C.B.R. values due to variation in the rate of loading should be continued.

III. Investigation of the various causes contributing to different types of cracking in bituminous surfacings should be carried out. In order to simulate the present day practice, to start with, the above investigation has to be made with regard to surface dressings under the following conditions :

(a) Due to weak subgrade

(b) Due to weak base course

(c) Due to weak wearing course

(d) Due to over-heated bitumen

(e) Due to stripping of bitumen in the presence of water

(f) Due to oxidation of bitumen on prolonged weathering.

Experimental stretches constructed at test track scale have to be subjected to both distributed as well as channelized traffic. Some of the sections have to be designed for long range studies under natural weathering conditions.

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" BITUMINOUS PENETRATION MACADAM "

By

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CONTENTS

SYNOPSIS

This Paper deals with an attempt to determine the quantities of binder and aggregates required for a full grout penetration macadam using hot binder. A laboratory stability test on the principle of extrusion was set up and utilised for determining the optimum quantity of binder required and the various quantities are worked out.

1. INTRODUCTION

It is well known that a bituminous surface treatment is the cheapest form of a bituminous road construction which acts only as a thin waterproofing without imparting any strength to the road structure as such. However, the stresses set up in the pavement immediately below the wheels by iron tyred traffic like bullock carts are very severe and are often sufficient to overcome the inherent stability of the water bound macadam pavement (which is essentially due to the stones being keyed tightly together with a slightly plastic binder), by forcing the

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stones out of position. A possible means of preventing this is to strengthen the structure of the pavement by introducing a bituminous binder into the road to hold the aggregates firmly in position.

The term 'bituminous macadam' designates a macadam road surface into which asphaltic material or refined tar is introduced either by mixing with the coarse aggregates before they are placed on the road or by pouring after the coarse aggregates are laid in place and allowing the binder to penetrate. However, this definition is not universal since, according to the American practice, bituminous macadam means only a penetration type of construction[1], whereas according to the British practice, it means a pre-coated type of construction[2]. Hence, in order to avoid the confusion it is proposed to use the terms "bituminous pre-coated macadam" and "bituminous penetration or grouted macadam" for the two different types of construction. Whereas the former forms either a base course or a binder course or a surface course in a road pavement according to the size and gradations of the stones employed, the latter always requires a cover coat.

When the binder is allowed to penetrate to the full depth of the stone layer it is called 'Full-grout' while, when it penetrates to only half the depth or less, it is known as 'Semi-grout'. It is the usual practice to have a consolidated thickness of 2 to 3 inches in the full-grout construction according to the traffic conditions and 2 inches in the semi-grout.

Bituminous penetration macadam, or grouted macadam ,as it is often called is probably the simplest way of producing an asphalt pavement, as the coating of the aggregate with the binder is carried, out in the road itself without the aid of mechanical mixers.

2. AGGREGATES FOR PENETRATION MACADAM

2.1. General

The success of a bituminous penetration macadam depends on the stability built up in the wearing course material by the binder that was poured into it and by the interlock of the aggregates. Undoubtedly, the wearing course with the maximum aggregate density, i.e., the stone layer built by closely graded aggregates so that minimum voids are left in it, will be the most stable one. But, such a layer of aggregates cannot be coated by the hot binder by just pouring it in for the obvious reason that the binder cannot flow into such small voids. Since the aggregates get coated only by the penetration of the binder into the layer, it is obvious that the aggregate layer should contain voids which are big both in total volume and in individual size. Hence, in order to strike at a balance between the stability and the aggregate density, the mineral aggregate for a bituminous macadam surface is normally of three gradings, given as below:

1.	The main wearing course or macadam material	coarse in size
2.	The choke material or key stone	intermediate in size
3.	The blotter or cover material for seal coat	fine in size

And it goes without saying that the aggregates should be hard and tough.

Normally, broken stones passing a $2\frac{1}{2}$ in. and retained on a $1\frac{1}{2}$ in. and passing a 2 in. and retained on a 1 in. square mesh form the wearing course material for a 3 in. and 2 in. thick penetration macadam respectively. The coarse aggregate is spread on the prepared base course in specified quantities so as to produce the required compacted thickness. After rolling and the first application of the bituminous binder, the intermediate sized stones, frequently designated as key stone and sometimes called as the choke stone, chips, splinters or peastones, of sizes from $1\frac{1}{2}$ in. to $\frac{3}{4}$ in. and from 1 in. to $\frac{1}{2}$ in. for 3 in. and 2 in. thick penetration macadam respectively, are spread uniformly in specified quantities and rolled until the surface becomes firm and tight. Then, to produce the seal coat, a second application of the bituminous material in specified quantities is made which is followed by the application of the cover material and the final rolling is carried out.

2.2. Quantity of Aggregates

It is essential that only the correct amount of aggregates should be used, because excess aggregates in the wearing course will lead to a thicker layer where the voids will tend to be less completely filled with bituminous material and with chips and fine stones.

The quantity of aggregates required for 3 in. and 2 in. thick penetration macadam pavements can be worked out in the following manner:

(a) **3 in. penetration macadam**

Assuming the per cent voids in the compacted layer as 40, it may be seen that the solid volume occupied by the aggregates alone will be

$$= \text{total volume} - \text{volume of voids cu. ft.}$$

$$= 100 \times \tfrac{1}{4} - 100 \times \tfrac{1}{4} \times \tfrac{40}{100} \text{ cu. ft.}$$

$$= 25 - 10$$

$$= 15 \text{ cu. ft.}$$

Assuming an average specific gravity of 2.7 for the aggregates, the total weight of the solid aggregate filling the volume of 15 cu. ft. is

$$= 15 \times 2.7 \times 62.4 \text{ lb}$$

$$= 2527 \text{ lb}$$

Assuming a bulk density of 85 lb per cu. ft., the total bulk volume of aggregate required is

$$= 2527/85 \text{ cu. ft.}$$

$$= 30 \text{ cu. ft.}$$

(b) **2 in. penetration macadam**

Working in a similar manner it can be seen that the quantity of aggregates required for the wearing course of a 2 in. penetration macadam will be about 20 cu. ft.

3. BINDER FOR PENETRATION MACADAM

3.1. Characteristics of the Binder

The important characteristic of the binder should be its capacity to penetrate into the compacted stone layer and hold the aggregates firmly against the disrupting forces set up by the traffic. Bituminous emulsions and cut-backs readily satisfy the first condition but take some time to cure before imparting the needed strength to the structure. However, the advantage in using these binders is that they are applied cold and that the construction can be done even in cold weather.

On the other hand, residual bitumens and refined tars have to be heated sufficiently before spraying on the aggregates.

Although the temperature of the hot binder is relatively high it will readily cool off to the temperature of the stone when sprayed on the cold aggregates, which will hamper its penetration into the layer, and the binder will be unable to flow freely into all the cavities. Hence, it is of paramount importance to choose a binder of "such a viscosity as will allow it to penetrate the aggregates before cooling. In this connection it must be remembered that the maximum permissible penetration for the bitumen is 65, which may be considerably reduced by the filler with which it is afterwards loaded"[4].

However, in a country like India where the summer temperature is very high, a still harder grade binder such as 30/40 penetration could be used without any disadvantage when the atmospheric temperature is above 100°F. At other times, binders of 60/70 or 80/100 penetration could be used.

When tar is employed as the binder, it is preferable to use as hard a grade as possible, such as R.T. 4 or R.T. 5, which will hold the aggregates more tightly and firmly than the softer grades by virtue of their higher viscosity.

It has been the experience elsewhere that the harder grades of binder have led to a greater success in the construction of bituminous penetration macadam pavements[1]. This suggests the use of harder grades of bitumen and tar cut-backs with suitable solvents.

3.2. Quantity of Binder

It is often erroneously held that in a penetration macadam the binder is supposed to fill in all the voids of the aggregates completely. This will be understood when it is considered that to fill up the 40 per cent of voids in the compacted layer, 10 cu. ft. of the binder by volume is required, in other words, 625 lb of binder for every 100 sq. ft., which is obviously too much for any pavement. Moreover, any free binder filling the voids which are sufficiently big in volume will do more harm than good by lubricating instead of binding the aggregates.

Since the purpose of the binder is to strengthen the structure of the pavement by holding the aggregates tightly it should penetrate and coat as much of the suface area of the aggregates as possible. However, it must be noted that the coating will be much thicker than in a premix type, because the coating is effected only by the free flow of the binder. Hence, the quantity of binder in a penetration macadam shoud be such as to impart maximum stability to the structure by coating the maximum surface area of the aggregates and such as not to leave excess of free binder in the voids which is likely to be drained off. With this in view, an attempt was made to determine experimentally the correct quantity of binder required for a bituminous penetration macadam as described in the following Sections.

3.3. Test Procedure

3.3.1. 3 in. penetration macadam

The stability of the penetration macadam was determined as the total load in pounds required to extrude a specimen of 2 ft 0 in. × 2 ft-0 in. × 0 ft-3 in. through a 5 in. diameter hole by a 2½ in. diameter plunger. The test specimen was prepared and tested in the following manner.

A steel tray of 2 ft-0 in. × 2 ft-0 in. × 0 ft-3 in. with a central hole of 5 in. dia. was placed on a cement concrete platform, and 1.2 cu. ft. of coarse aggregates passing 2½ in. and retained on 1½ in. square mesh (I. R. C. Std. 2 in.) were spread on the tray, corresponding to a rate of 30 cu. ft. per 100 sq. ft., Photo 1. A galvanized iron sheet was spread over the aggregates and a 1 ton vibratory roller was brought on the G. I. sheet, Photo 2. Rolling was done for about 3 minutes till the aggregates were well compacted and the surface was firm and uniform.

The binder was heated to its application temperature and was hand-sprayed on the aggregates in specified quantity by means of a perforated tin, care being taken to spray the binder uniformly all over the aggregates. Asphaltic bitumen 80/100 and road tar RT-5 were employed in this investigation. The sprayed surface was allowed to cure for 24 hours.

After curing, the tray was gently lifted up and supported below by means of concrete blocks about 3 in. high. A portable loading frame was fixed over the specimen (as shown in Photo 3) and the specimen was centred. A plunger of 2½ in. dia. was placed at the centre of the sprayed surface, where the voids were filled with dry, coarse sand over which a thin film of plaster of paris was applied, for uniform seating of the plunger. After the plaster of paris had set, the load was applied by means of a proving ring at the rate of 0.25 in. per minute. The maximum load at failure in pounds was recorded as the stability of the specimen. A series of tests was repeated for the same binder content and the average of the best three values was taken as the stability for the particular rate of binder. The tests were carried out at an atmospheric temperature of about 28°C.

The rates of application of the binder tried were: 50, 100, 125, 150, 165, 185, and 200 lb per 100 sq. ft. The test results are given in Tables 1 to 4.

TABLE 1

Stability values for 3 in. penetration macadam using bitumen 80/100

Quantity of binder lb per 100 sq. ft.	Stability lb
0	200
50	350
100	600
125	725
150	1175
165	1400
185	1500
200	1575

TABLE 2

Ratio of cohesive resistance/drained binder for 3 in. penetration macadam using bitumen 80/100

Binder used lb per 100 sq. ft.	Binder drained lb per 100 sq. ft.	Effective binder lb per 100 sq. ft.	Cohesive resistance lb	Cohesive resistance/ effective binder
50	—	50	150	∝
100	—	100	400	∝
125	—	125	525	∝
150	3	147	975	325
165	8¼	156¾	1200	145
185	18½	166½	1300	70
200	36	164	1375	36

TABLE 3

Stability values for 3 in. penetration macadam using tar R.T. 5.

Quantity of binder lb per 100 sq. ft.	Stability lb
0	200
100	500
125	675
150	1000
165	1175
185	1250
200	1250

TABLE 4

Ratio of cohesive resistance/drained binder for 3 in. penetration macadam using tar R.T. 5

Binder used lb per 100 sq. ft.	Binder drained lb per 100 sq. ft.	Effective binder lb per 100 sq. ft.	Cohesive resistance lb	Cohesive resistance/ drained binder
100	—	100	360	∝
125	—	125	475	∝
150	6	144	800	133.3
165	22	143	975	44.3
185	31	154	1050	33.8
200	50	150	1050	21

After the stability was determined, the tray was emptied of the aggregates and weighed with the binder sticking to the plate. The quantity of the binder that had drained to the base plate was found out by substracting the tare weight of the tray from this weight. The quantity of effective binder was obtained by deducting the quantity of the drained binder from the total quantity of the binder that was used, Table 2.

3.3.2. 2 in. penetration macadam :

A similar test was carried out to determine the quantity of binder required for a 2 in. penetration macadam. In this case the size of the tray was 1 ft-6 in. × 1 ft-6 in. × 0 ft-2 in. with a central hole of 4 in. in diameter and the plunger used to extrude the specimen was 2 in. in diameter, otherwise the preparation and the testing of the specimen were the same. About $\frac{1}{2}$ cu. ft. of aggregates passing 2 in. sieve and retained on 1 in. sieve (I.R.C. Std. $1\frac{1}{2}$ in.) was used correspoding to 20 cu. ft. per-100 sq. ft.

The following rates of application of the binder were tried: 50, 70, 90, 110, 130, and 150 lb per 100 sq. ft. The test data are given in Tables 5 to 8.

3.4. Discussion of Test Results

3.4.1. Before any attempt is made to interpret the test data it is significant to recall the shortcomings of this test procedure so as to appreciate the extent of the validity of the test results. Since the prepared 'specimen' for test comprises big sized irregular aggregates of varying shape spread on a large area, it is

TABLE 5

Stability values for 2 in. penetration macadam using bitumen 80/100

Quantity of binder lb per 100 sq. ft.	Stability lb
0	225
50	425
70	550
90	625
110	775
130	950
150	1250

TABLE 6

Ratio of cohesive resistance/drained binder for 2 in. penetration macadam using bitumen 80/100

Binder used lb per 100 sq. ft.	Binder drained lb per 100 sq. ft.	Effective binder lb per 100 sq. ft.	Cohesive resistance lb	Cohesive resistance/effective binder
50	—	50	200	∝
70	—	70	325	∝
90	—	90	400	∝
110	3	107	550	183
130	11	119	725	66
150	22	128	1025	46.5

TABLE 7

Stability values for a 2 in. penetration macadam using tar R.T. 5

Quantity of binder lb per 100 sq. ft.	Stability lb
0	225
70	450
90	575
110	725
130	900
150	1200
180	1300

impossible to produce specimens of identical nature. Hence, though the same quantity of binder and aggregates were used to prepare 'identical' specimens, their behaviour in an extrusion test is bound to show some difference because of the position of interlock of the aggregates being dissimilar in different

Photo 1.

$1\frac{1}{2}$ in. loose aggregates in steel tray of size $2' - 0'' \times 2' - 0'' \times 0' - 3''$

Photo 2.

Rolling with the vibratory roller in progress

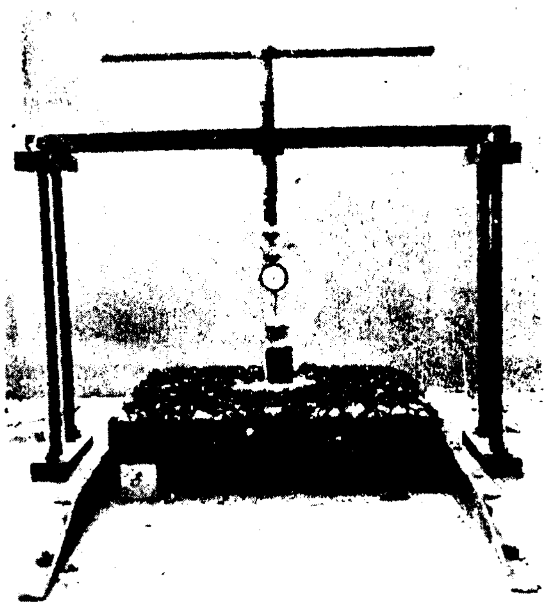

Photo 3.
Test assembly with the portable loading frame and proving ring

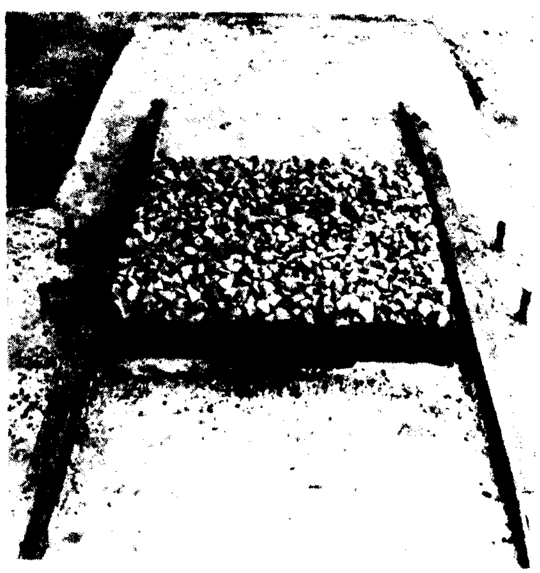

Photo 4.
Keystones spread and rolled over the binder-sprayed -

TABLE 8

Ratio of cohesive resistance/drained binder for 2 in. penetration macadam using tar R.T. 5

Binder used lb per 100 sq. ft.	Binder drained lb per 100 sq. ft.	Effective binder lb per 100 sq. ft.	Cohesive resistance lb	Cohesive resistance /drained binder
0	—	—	—	—
70	—	70	225	∝
90	—	90	350	∝
110	3.5	106.5	500	142
130	11	119	675	61
150	33	117	975	29
180	50	130	1075	21.5

specimens. For this reason, the test was repeated for a number of times and the average of three near-about identical test results was taken for the purpose of their interpretation.

The loading area under the plunger (4 in. and 5 in. dia.) is more or less equivalent to the area of contact of a loaded vehicle and, since the area is small compared to the area of the steel tray, the question of confinement within the tray is easily overcome.

As regards the saw-tooth bond that is normally obtained in the field between the macadam layer and the base, in the laboratory test it is purposely eliminated, not only with a view to eliminate the wide variations of this effect but more so to avoid the effect of the binder drained into the base. For, the object of this investigation is only to study the increase in the strength of an individual compacted layer of stones, as a result of increasing quantities of the grouted binder.

Hence, though the test method is a purely empirical one, the test data could be interpreted in a rational manner to some extent as shown in the following Section subject to the above mentioned limitations.

3.4.2. In Fig. 1 is drawn the relationship between the quantity of binder employed and the stability value in lb. From the Figure it can be seen that even the uncoated aggregates have

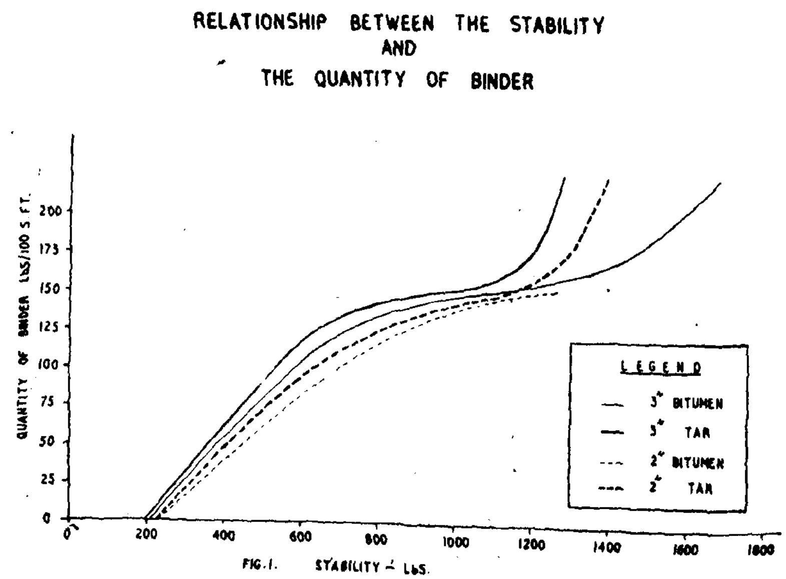

FIG. 1.

a stability of 220-225 lb indicating the interlocking resistance of the coarse aggregates when compacted. With the subsequent spraying of the binder, the stability increases gradually up to a certain optimum value beyond which it is seen that the increase in stability for a corresponding increase of binder is lower than in the initial stages.

This indicates that the stability of bituminous penetration structure is made up of two distinct factors, viz.,

(a) resistance due to interlock of the compacted aggregates which can be assumed to be constant for a given aggregate, and

(b) resistance due to cohesion brought into play by the binder and which is variable (obtained by subtracting the interlocking resistance from the stability value).

Normally in a water-bound macadam, this cohesion is supplied by the plastic filling material employed whose cohesive

strength is extremely variable depending on moisture and other conditions. The importance of high cohesive strength in the layers immediately below the wheels of traffic has been very well brought out by various investigations such as shown in the A.A.S.H.O. test report [14]. And so it is obvious that where only a thin *surface treatment* is given, the macadam layer below it should possess as high a cohesive strength as possible.

This poses the question as regards the quantity of bitumen that is necessary to supply this cohesion. It is well known that the cohesive resistance due to a bituminous binder in a road structure is a function of the surface area coated, the thickness of the coated film and the consistency of the binder. Assuming that the last two factors are constant in the test, it follows that the increase in cohesion is due to an increased coated area of the aggregates as a result of increased quantities of binder. But then as can be seen in Figure 1, there is an optimum value at which the surface area coated is maximum and beyond which use of an excess of binder does not materially increase the strength but, on the other hand, only gets drained.

It can be seen from Fig. 2 that as the binder quantities are increased beyond a certain value, the amount of binder drained goes on increasing and also from Tables 2, 4, 6 and 8 it can be

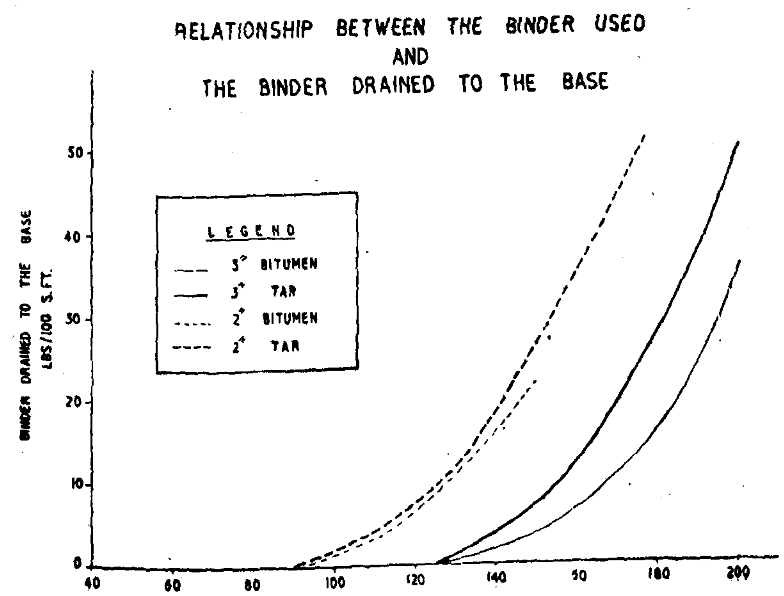

FIG. 2.

seen that, beyond a certain value, the effective binder (the quantity of binder employed minus the quantity of drained binder) remains more or less constant. Though it may be said that at very low quantities of binder there is practically no loss in the form of drained binder, yet the adequate stability may not be obtained, for which reason the maximum finite value obtained for the ratio of the stability value to the drained binder will also have to be considered, Fig. 3.

RELATIONSHIP BETWEEN THE QUANTITY OF BINDER AND THE RATIO OF STABILITY / DRAINED BINDER

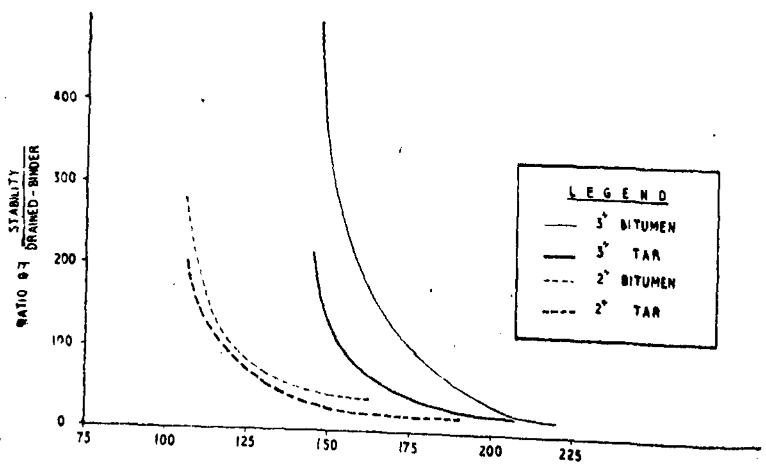

FIG 3 QUANTITY OF BINDER LBS/100 S. FT.

So, in order to fix the quantity of binder necessary, the criteria should be as under:

(a) A reasonably high value of stability should be aimed at.

(b) There should be a minimum loss in the form of drained binder.

(c) A reasonably high value for the ratio of stability of the drained binder should also be aimed at.

Bearing in mind the above criteria, it can be seen that for a 3 in. penetration macadam with bituminous binder, the quantity of bitumen of 140-150 lb per 100 sq. ft. would be quite satisfactory, and for a 2 in. penetration macadam, the figure would be about 110 lb per 100 sq. ft.

As regards using tar as a binder, it can be seen that the behaviour in general is similar to that with bitumen. However, for both the 3 in. and the 2 in. layers, the stability values are correspondingly lower than for bitumen. Also the corresponding quantity of drained binder is more in the case of tars than in bitumens. These are obviously due to the lower viscosities of the tars. Because of this very same reason, the same quantities are indicated for both, in spite of the higher specific gravity of the tar as against that of bitumen.

In Table 9, it can be seen that the prevalent practices advocate extremely varying quantities of binder such as 120-220 lb per 100 sq. ft. for a 3 in. and 100-180 lb for a 2 in. macadam. By adopting the figures indicated in this Paper, it is felt that a saving to the tune of 15-20 lb per 100 sq. ft. can easily be effected.

TABLE 9

Quantity of binder used for the wearing course in the penetration macadam in lb per 100 sq. ft.

Name of authority	Binder for 3 in. macadam	Binder for 2 in. macadam
1. Bureau of Public Roads, U. S. A.	165	—
2. Asphalt Institute, U. S. A.	220	—
3. State of Madras	165	110
4. State of Bombay	130–170	140–168
5. Central P. W. D.	130	100
6. Burmah Shell	150	100
7. Standard Vacuum Oil Co.	140 (2½ in. thick)	112
8. Caltex India Ltd.	165	—
9. Shalimar Tar Products Ltd.	—	180

4. KEYSTONES

4.1. After application of the hot binder for grouting, the keystones are spread to secure a good interlock at the surface. The quantities and sizes should be such that the voids of the macadam layer are filled sufficiently. Excess of aggregates will only remain loose and be whipped off by traffic.

A lower quantity would result in the wearing off of the macadam stones instead of the keystones and the seal-coat. Again, in order to produce a non-skid and less rough texture, grading of the keystone has also been suggested.

So in order to assess the quantities and sizes of keystones required for a 3 in. and 2 in. macadam, the following test was adopted.

4.2. 3 in. Penetration Macadam

After spraying a suitable quantity of binder over the rolled macadam in the tray, aggregates passing 1½ in. and retained on 1 in. sieve were placed by hand in the voids to fill up the bigger sized voids and those passing 1 in. and retained on ¾ in. sieve were placed in the smaller sized voids. After completely packing the voids, the small sized aggregates were spread on the macadam course material so as to make shoulder to shoulder contact with the aggregate, already packed, so that all the sprayed area was covered completely, Photo 4. Then the whole surface was rolled and, where necessary, further quantity of the small sized aggregate was placed by hand. The quantity of the keystones thus used was then determined.

It was found that the keystones necessary to fill up the voids of the wearing course and to spread on the whole surface was about 6.5 cu. ft. per 100 sq. ft. of aggregates passing 1½ in. and retained on ¾ in. sieve, which was made up of 1 part of 1½ to 1 in. and 5 parts of 1 to ¾ in. sizes.

4.3. 2 in. Penetration Macadam

Carrying out a similar test, it was found that the keystones necessary to fill up the voids of the wearing course and to spread on the whole surface were about 5 cu. ft. per 100 sq. ft. of aggregates passing 1 in. and retained on ½ in. sieve, which was made up of 1 part of 1 to ¾ in. and 5 parts of ¾ to ½ in. sizes.

5. SEAL COAT

No attempt is made in this investigation to include seal coating which should be given on a penetration macadam course after the application of the keystones to suit the particular type and volume of traffic and to give the pavement a smooth and non-skid surface. This can be achieved by adopting any suitable specification for a seal coat.

6. CONCLUSIONS

1 The results of the laboratory investigations seem to indicate that the quantities of aggregates and bituminous materials required for the wearing course of 3 in. and 2 in. penetration macadam constructions of hot-grouted process to yield a stable and economical pavement should be as given in Table 10.

2. It appears that asphaltic bitumen or road tar can be used as binder in equal quantities by weight in penetration macadam constructions.

TABLE 10

Quantities of aggregates and binder, bitumen or tar, for wearing course of penetration macadam

Compacted thickness inches	Aggregates for wearing course: Size: Passing sieve inches	Aggregates for wearing course: Size: Retaining sieve inches	Aggregates for wearing course: Quantity cu. ft. per 100 sq. ft.	Keystones: Size: Passing sieve inches	Keystones: Size: Retaining sieve inches	Keystones: Quantity cu. ft. per 100 sq. ft.	Binder for wearing course lb per 100 sq. ft.
2	2 (I.R.C. 1½ in.)	1	20	1	½	5	110
3	2½ (I.R.C. 2 in.)	1½	30	1½	¾	6.5	150

7. ACKNOWLEDGMENT

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INFORMATION SECTION

"STRESS AT A POINT WITHIN A SEMI-INFINITE ELASTIC MEDIUM SUBJECTED TO A SURFACE VIBRATORY LOAD"

By

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SYNOPSIS

The object of this Paper is to present the stress-strain relationship in soils under static loads and also the elastic and plastic deformations under vibratory loads. The Paper gives the stress condition at a point in a soil mass subjected to a surface vibratory load. This aspect has a large bearing on the pressure generated by mechanical compaction equipment used in the field and these factors are discussed.

The Paper also gives a description of the 'Degebo' type of vibration generator and srain gauge pressure pick-up both designed and fabricated for research work at the Soil-Mechanics Laboratory of the Indian Institute of Science.

1. STRESS-STRAIN RELATIONS FOR SOIL

For a soil specimen subjected to a constant lateral stress σ_x and a vertical increasing stress σ_y, if a graph of σ_y against the strain in the vertical direction is plotted, it may be observed that this curve is more or less non-linear one throughout its length. The stress value reaches to a maximum and falls down and the type of curve obtained depends on the type of soil. The peak stress is assumed to be the failure stress. The slope of the more or less straight portion at the initial range is referred as the modulus of deformation. In the lower ranges of the stress-strain curve, the strains are reversible. This is the range of elastic deformation. But on applying the stress to a little less than the ultimate load, the strain caused is no longer reversible and this is the range of plastic deformation.

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If a soil specimen is repeatedly loaded and unloaded a number of times, keeping the value of the maximum load a little less than the peak load, and if stress-strain relationship is plotted for each cycle of load, the modulus of deformation starts increasing gradually and the permanent strain for each load repetition appears to be constant after a number of load repetitions. This behaviour is shown in Fig. 1.[1] Now if similar repetitional loads are applied on the surface of a soil-medium,

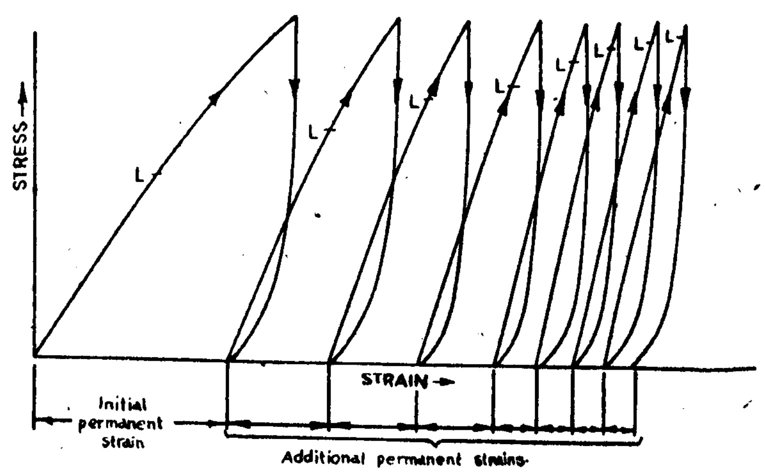


Fig. 1. Typical stress-strain relationship for repeated application of compressive load to sandy clay

the general trend of the stress-strain graph will be more or less the same as that shown in Fig. 1. The effect of this load repetition is to increase the bulk-density of the soil. Confining the study in this Paper to sand alone, there are three possibilities for the granular soils to have their grains rearranged or deformed and they are explained in Fig. 2.[2]

The three conditions are:

(a) The elastic condition

(b) The plastic condition

(c) The reallocation condition.

If a pressure cell is embedded in the soil and if the elastic condition prevails, the stress on the surface generates a pressure which is recorded by the cell and this pressure vanishes completely on removal of stress from the surface. But under the plastic condition, the pressure recorded by the cell does

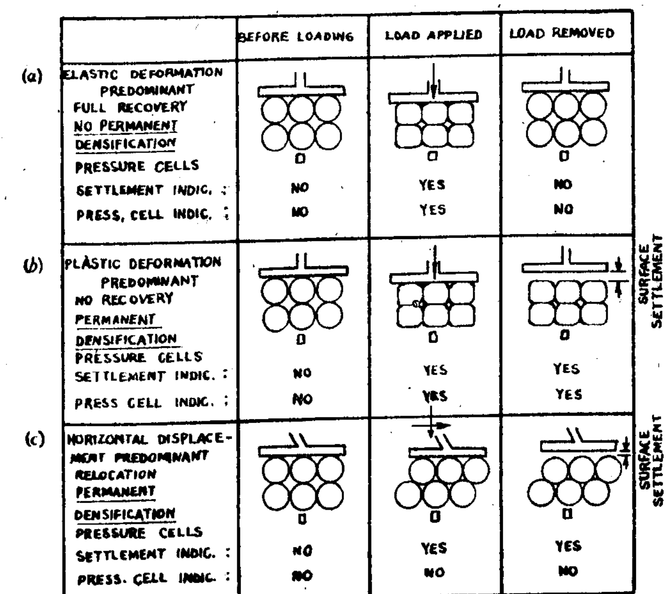


FIG 2. Diagram Illustrating Surface Settlement, Pressure Cell Indication, and Soil Density (Spheres of Equal Diameter Assumed) (AFTER - BERNHARD & FINELLI.)

not vanish despite the fact that the stress from the surface is removed. If the reallocation condition prevails due to horizontal displacement of soil-grains, the pressure cell does not record any stress.

Assuming that the soil mass behaves elastically, analysis has been done by Boussinesq for the pressure distribution inside the semi-infinite elastic medium due to a surface load. This has been done by assuming a stress function to suit the boundary conditions and also satisfying the biharmonic equation. The simplified form of Boussinesq's equation reads as:

$$\sigma_z = \frac{3Q}{2\pi Z^2} \cos^5 \psi \qquad \ldots\ldots (1)$$

Where σ_z is the intensity of vertical stress at a depth Z below a point surface load Q and ψ is the angle between the radius vector and the vertical line through the surface load. It should, however, be borne in mind that this equation has a limited application for granular soils.

From the above equation, it is obvious that the stress condition inside the soil does not depend on the modulus of elasticity and the density of the medium. It is possible to draw contours of equal vertical stress called the bulbs of pressure, which for a perfect elastic medium and for point loads are spheres. These spheres are tangential to the surface of the medium.

2. EFFECT OF DENSITY ON THE STATIC PRESSURES AT A POINT

It has been observed by some investigators[3] that, if a horizontal section is taken and the pressures at a point are plotted for different densities of soil, the pressures recorded are much more in the case of loose soils than in the case of dense soils. But what has been shown by Whiffin[4] contradicts this. His finding, while dealing with rollers, is that the peak pressure transmitted under a roller increases with density. The effect of density does not arise in the case of Boussinesq equation and hence the influence of density needs further verification. However, in the present context, we may take it for granted that the change in density alters the stress at a point. Whether this change is an increase or decrease would not matter as we are dealing with the combined effects of the three aspects explained in the next Section.

3. EFFECT OF A SURFACE VIBRATOR ON THE DENSITY OF SAND

The operation of a vibrator at the surface of sand medium definitely causes re-arrangement of grains of the soil, because each individual particle is excited into oscillation during the operation of the vibrator. If a vibrator is kept on the surface of a sand bed and operated, there is considerable variation in the density of the medium and the density can be accurately measured by two methods: by the use of a thin walled sampling tube and the other by a probing device. Converse[5] has shown by these methods that the density of sand is a maximum at a depth approximately equal to the width of the vibrator base.

Fig. 3 is a typical record of the contours of equal density before and after run.

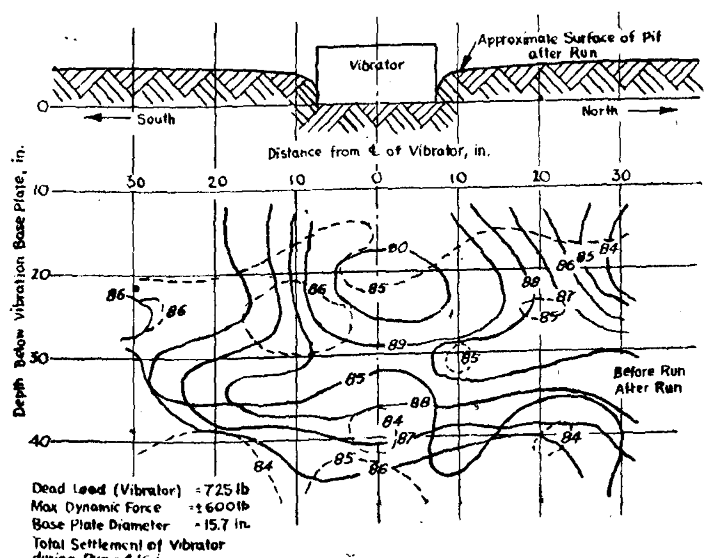


FIG. 3 Curves of Equal Dry Weight—Per Cent of Modified AASHO.
Vertical section through vibrator center line before and after run 26. (AFTER-CONVERSE.)

The pressure variation at a point inside the soil mass due to a surface vibrator can occur due to the following three causes:

(a) Due to the variation of density which is considerably affected by the operation of the vibrator.

(b) Due to the settlement of the vibrator, which in turn causes nearness of the load to the pressure cell (neglecting the displacement of the pressure cell itself).

(c) Due to the plastic deformation which can be a predominant factor. The effect of this is that the cell cannot come back to zero-reading at the end of each load cycle and the succeeding cycle causes an addition of stress to the residual stress of the earlier cycle.

There are many other conditions like the boundary effects, the discontinuity at the cell region, the arching effects, and the sensitivity of the cell used. All these are negligible or can be made as small as possible while the above three are prominent.

4. PRESSURES GENERATED BY THE COMPACTION EQUIPMENT

To record the dynamic pressure generated by a compaction equipment three types of gauges are available :

(a) Piezoelectric gauge

(b) Acoustic gauge

(c) Differential pressure cell.

The piezoelectric gauge depends on the principle that a piezoelectric substance like quartz crystal when subjected to a stress generates a voltage.

The acoustic gauge depends on the principle of a vibrating wire whose length varies due to the stress on a deflecting disc which in turn causes change in natural frequency.

The differential transformer is used in differential pressure cell. The movement of a membrane causes movement of a soft iron core which in turn creates a change in voltage.

Whiffin used acoustic gagues to measure the dynamic stresses generated by the movement of a compacting device, like the roller on the surface of the soil, keeping the pick-ups at different depths. One of the typical records of Whiffin's experiment is shown in Fig. 4. Some of the conclusions reached are:

(a) The peak pressure recorded increases with the number of passes and attains a limiting maximum value.

(b) The dry density of the soil attains a maximum value when the peak pressure recorded is maximum which in turn depends on the number of passes.

These follow from many test runs made for the different types of compaction equipments (like smooth wheel roller, power rammer, frog rammer, etc.) and for different types of soils.

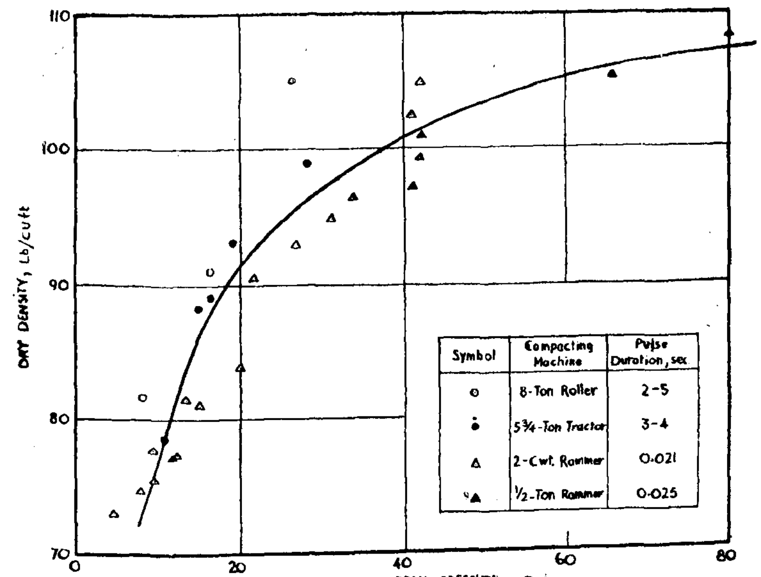


FIG. 4. Density-Pressure Relationship for Silty Clay, Moisture Content Approximately 15 to 16 per cent (AFTER- WHIFFIN)

5. INVESTIGATIONS AT THE INSTITUTE

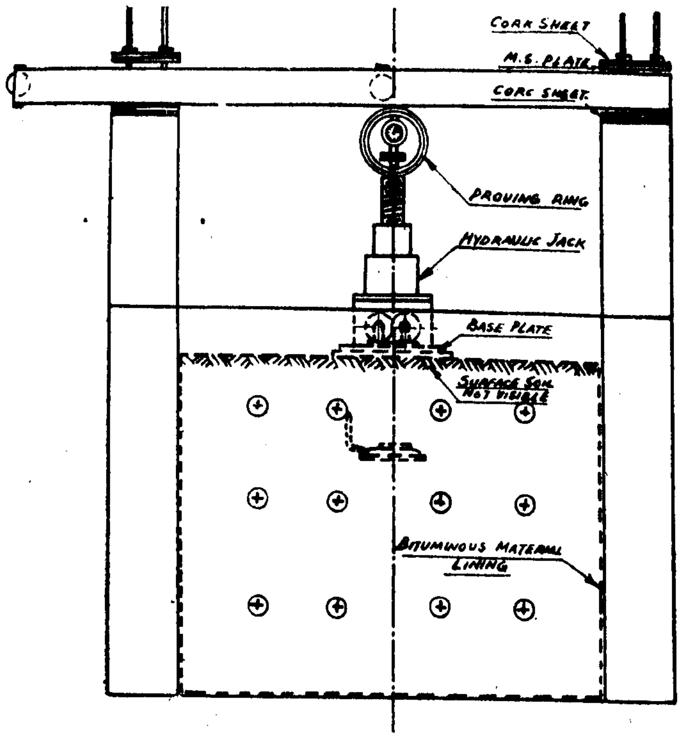
In the light of the above findings, the following aspects are under investigation.

1. Does the pressure increase as a function of time inside the soil mass due to a stationary vibratory load?

2. Has the area and shape of the surface base plate any effect on the stress at a point inside the soil mass?

It should now be pointed out that studies on instantaneous pressures are not aimed at. But the sum total of the effective increase or decrease of static pressure due to the increase of density, due to the settlement of the vibrator, and due to the possible plastic condition is aimed at and the soil chosen for this study is only sand as this does not present many difficulties to compact in a standard manner.

(a) **The details of the set-up.** A brick masonry tank of $4\frac{1}{2}$ ft $\times$ $4\frac{1}{2}$ ft $\times$ 4 ft was constructed. On two opposite walls two small pillars of cement concrete were constructed with proper reinforcements. A channel section is fixed over these two pillars. Fig. 5 shows the completed set-up. Inside the tank a bitumen lining is given so that the reflection of the waves from the solid boundary is decreased considerably, when the vibrator



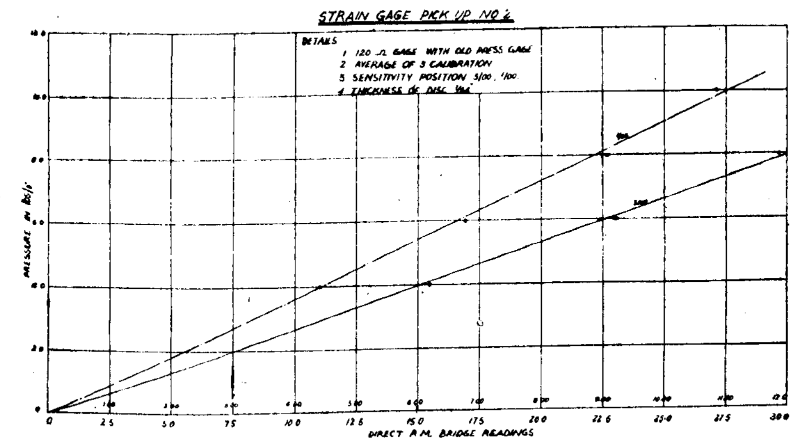
SET-UP FOR SOIL VIBRATION

FIG. 5.

starts operating. Sand passing 7 sieve (B.S.S.) and retained on 72 sieve (B.S.S.) was filled in. After laying every 500 pounds of sand it was levelled and tamped by a tamping device of 9 pounds weight distributed over an area of 6 in. × 18 in. This was dropped from about 3 in. height.

(b) **The pressure cell and its calibration.** A fairly sensitive type of pressure pick-up was designed by using a strain gauge on a very thin circular aluminium disc, simply supported on its edges. Each one of these cells was calibrated by measuring the variation of resistance due to a uniform compressed air pressure by an A-C measuring bridge. This bridge has the faci-

lities for the variation of resistances and capacitances to compensate the differences in amplitude and phase components of the active and temperature compensating gauges. A typical calibration chart for one of the pick-ups is given in Table 1, and this is plotted as a graph in Fig. 6, showing the relation



between the surface uniform pressure and the corresponding reading on the measuring bridge. A detailed description of the cell and its calibration are given[7].

TABLE 1

Calibration of pressure pick-up No. 2

Sensitivity position 1/00; Disc: 1/64 in.

Pressure in psi	I	II	III
0	0.00	0.00	0.00
4	4.50	4.50	4.70
6	7.00	6.80	6.50
8	9.20	9.00	9.00
10	11.00	11.00	10.50

(Average of the above three is taken)

(c) **The vibrator.** A Degebo type of vibration generator was designed and fabricated in the workshops. There are two gearwheels rotating in opposite directions and two eccentric masses are so situated that the horizontal components of the centrifugal forces vanish and the vertical components of this force will give a pure sine wave. These wheels are capable of generating about 1000 pound weight of maximum dynamic load. The static weight of the vibrator is 70 pounds. A one H.P. direct current motor with a 6 feet flexible shaft drives the vibrator. The D.C. motor is of separately excited type and is supplied by a rectifier with a Thyratron Control and the speed can be easily regulated by a manual controller capable of smooth changes in speed.

(d) **The base plates and the static loading device.** The following size base plates were tried in the laboratory:

1. Rectangle 12 in. × 18 in.
2. Rectangle 9 in. × 18 in.
3. Rectangle 9 in. × 13½ in.
4. Square 12 in. × 12 in.
5. Circular 12 in. diam.
6. Octagon of 6 in. sides

Three different static loading conditions were tried and these are:

1. 1686 lb (30 st)
2. 2218 lb (40 st)
3. 2778 lb (50 st)

Hitherto the approach on the problems of dynamic load or static load has been to keep the stress intensity on the base plate to be a constant, for comparison. But based on some observations[6] it was proposed to keep the total load on all base plates as constant. But by the present set-up different combinations of static and dynamic loads are possible.

(e) **Pressure increase as a function of time.** A strain gauge pressure pick-up was placed at a distance of 9½ in. from the surface of the bed and placing the plates, jack, etc., on the surface the vibrator was made to run at 2000 R. P. M. in about 10 seconds generating about 375 pounds of maximum dynamic load. The pressure was recorded in terms of the readings of the measuring bridge at one minute intervals till 5 minutes after starting the vibrator. As an additional precaution for constancy of speed of rotation of the vibrator, a Stroboscope was also used throughout the experiment.

When the vibrator starts operating there is a rapid settlement of the base plate and this causes the static load to come down in value, but immediately this is made good by operating the jack so that the dial gauge of the proving ring always oscillates on the mean position.

After running the experiments, the loads and the base plate were removed and the sand bed was thoroughly disturbed by raking to a depth of 6 to 8 inches till the pressure pick-up reading comes back to zero and the surface is again levelled and tamped. Any discrepancy in the initial reading is corrected and the set-up was kept idle for the next fifteen minutes. It was noticed that there was very little difference in the level of the sand after this process. Fig. 7 shows the pressure increase with reference to time. It may be mentioned here that the calibration chart of the pressure cell is more or less linear in the initial ranges

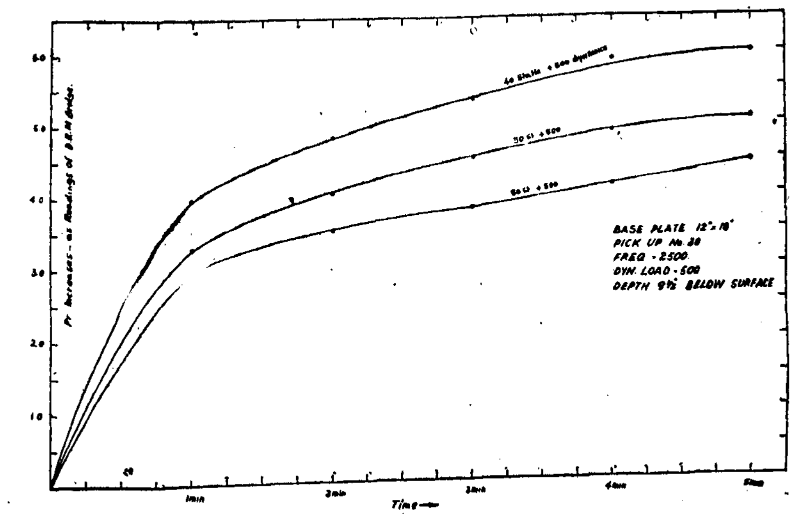


FIG. 7.

and for the load ranges measured; the correction for the initial reading from experiment to experiment, which is sometimes positive and sometimes negative, does not matter at all because these are extremely small. But the only disadvantage of the pick-up is that there is a change in the initial reading for a pure static load from experiment to experiment, but the increase is more or less the same for all the experiments. Figs. 7 and 8 show the trend of pressure increase with reference to time for two different base plates for three static loads and a definite dynamic load.

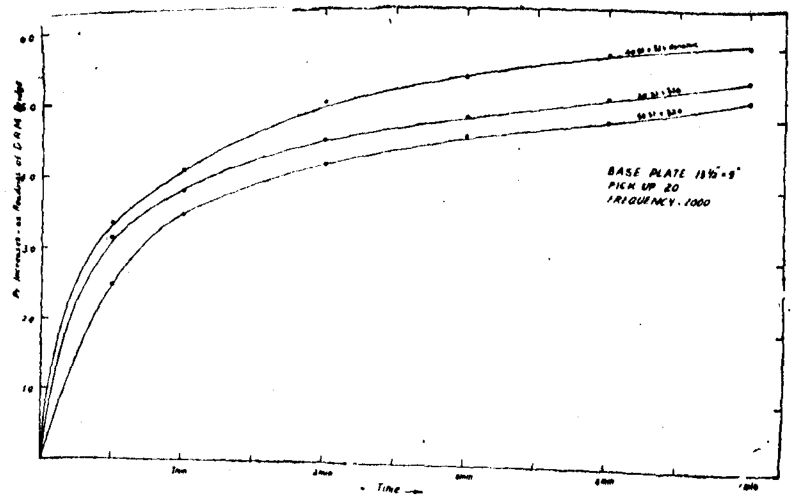


FIG. 8

(f) **Conclusions.** 1. It appears obvious that the pressure increase depends on time and the major pressure increase occurs in about 1 to 1½ minutes after starting the vibrator and attains a peak value after 5 minutes. The further increase in the next 15 minutes is negligibly small.

2. There appears to be a particular combination of static and dynamic loads for a base plate which gives maximum increase of pressure at the point and for higher or lower combination of total loads the pressure increase appears to be less.

3. These results pertain to one point which lies directly below the centre of the base plate. The state of stress at other points, the effect of frequency on the pressure increase, the distribution pressure increase in the horizontal and vertical planes are still under study.

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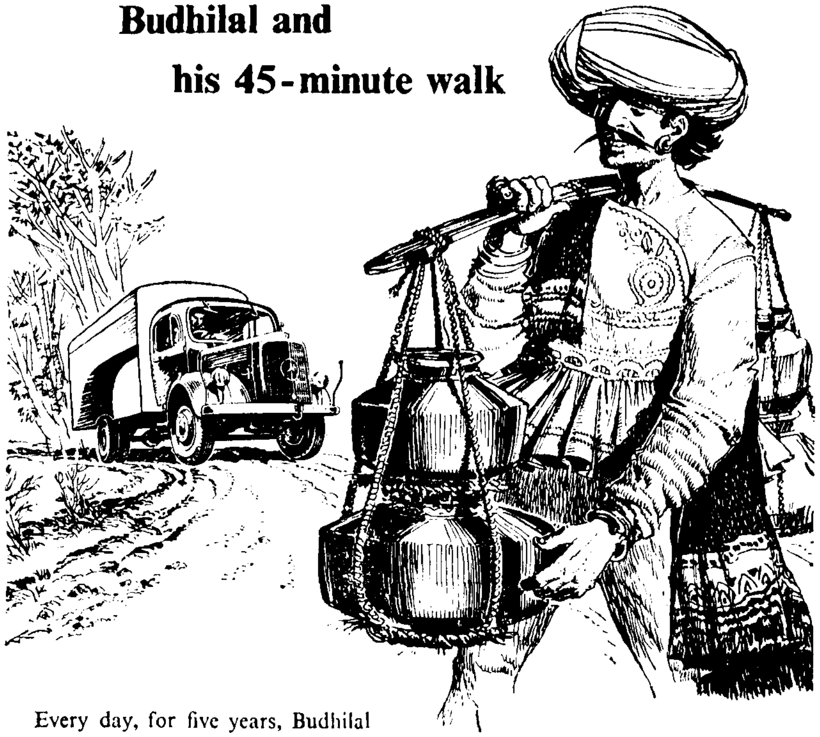

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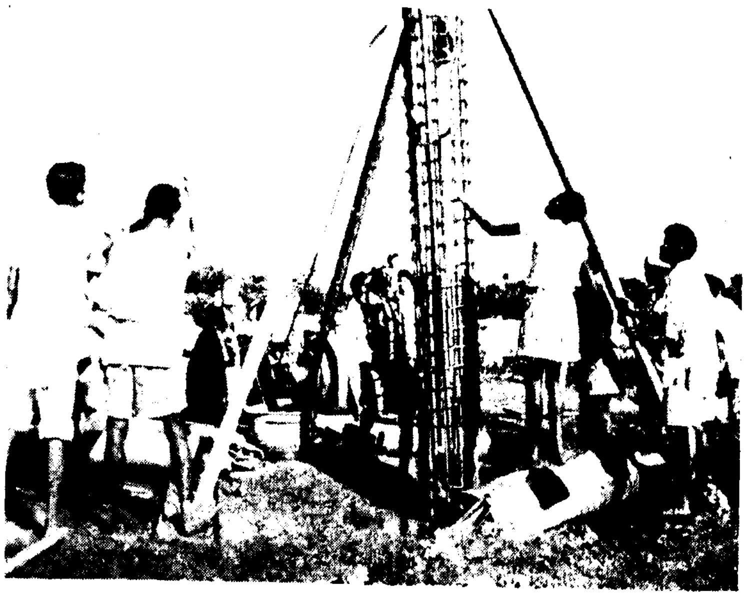

The above view shows work in progress on the construction of 19" diameter Piles For DHAMDACHA Bridge in Navsari Dangs Division for Bombay P.W.D.

Undertaken by

THE CEMENTATION COMPANY LIMITED

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it came out of a tree

A school boy, writing an essay on Dunlop, said: "When Dunlop first came into being it came out of a tree. There were two men walking in the forest one day one man kept throwing his knife in the trees. He came to one tree and through his knife in it and something sticky came out. After a while it stopped coming out and at the bottom of the tree there was a big ball of rubber. After a while they started playing football with it and that is how it came into being. I will now tell you what Dunlop is used for..."

At this stage, perhaps, we should rescue our readers from school boy language and list the products that Dunlop make in India...

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