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Paper No. 167

THE DESIGN OF SMALL BRIDGES AND CULVERTS

By

GOVERDHANLAL

Paper No. 167 2 31

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SYNOPSIS

The discussion begins by stressing the importance of collecting sufficient data before designing is started. Methods for calculating flood discharges from the catchment area and rainfall, from the conveyance factor and slope of the channel, and from flood marks on existing structures have been outlined and suggestions made for fixing the design discharge (Articles 4-7).

After giving an outline of the "Lacey's Theory of Flow in Incoherent Alluvium", the procedure for designing the linear waterway and estimating the normal depth of scour for alluvial streams has been included. This is followed by a discussion on the normal scour depth of quasi-alluvial streams and the effect of contraction on the scour depth. Rules for calculating the maximum depth of scour and designing the depth of foundations have been included. Their application has been illustrated with worked out examples.

Notes on fixing the length and number of spans and on some important structural details of minor bridges come next.

The hydraulics of flow through bridges and its application to the calculation of discharges through existing bridges as well as of afflux through proposed bridges have been dealt with in some detail. Examples have been worked out in illustration.

Brief notes on how to deal with over-flows have been added. The discussion concludes with Articles on the feasibility of pipe and box culverts and the hydraulics of designing them.

Six charts have been added at the end to facilitate routine computation.

INTRODUCTION

Road project estimates prepared for the Five-Year Plan are sometimes based on insufficient data and this causes delay in sanctioning them. While speed in execution of the Plan is essential, the quality of work has to be maintained.

The designing of a culvert or a small bridge is not a small matter to be despised by engineers. It must be remembered that the number of culverts and small bridges in any road project is so large that, together, they account for a considerable portion of the total expenditure.

The object of this Note is to draw attention to some aspects of designing culverts and small bridges for road projects. What is said here might appear simple and common place but experience has shown that serious mistakes are made in these simple matters and that leads to waste through unsafe and uneconomical designs.

Observations in this Paper apply only to small bridges and culverts. Larger structures require more detailed design procedure.

ARTICLE 1

SITE INSPECTION

1.1. Selection of Site. Where there is any choice, select a site :

(i) which is situated on a straight reach of the stream, sufficiently below bends ;

(ii) which is so far away from the confluence of large tributaries as to be beyond their disturbing influence ;

(iii) which has well defined banks ; and

(iv) which makes approach roads feasible on the straight.

1.2. Existing Drainage Structures. If, by chance, there is an existing road or railway bridge or culvert over the same stream and not very far away from the selected site, the best means of ascertaining the maximum discharge is to calculate it from data collected by personal inspection of the existing structure. Intelligent inspection and local enquiry will provide very useful information, namely, marks indicating the maximum flood level, the afflux, the tendency to scour, the probable maximum discharge, the likelihood of collection of brushwood during floods, and many other particulars. It should be seen whether the existing structure is too large or too small or whether it has other defects. All this should be carefully recorded.

1.3. Inspection should also include taking notes on channel conditions from which the silt factor and the coefficient of rugosity can be estimated.

ARTICLE 2

THE ESSENTIAL DESIGN DATA

2.1. In addition to the information obtained by personal inspection of an existing structure, the design data described in the following paragraphs have to be collected. What is specified here is sufficient only for small bridges and culverts. For larger structures detailed instructions contained in the IRC Bridge Code, clauses 100-101, should be followed.

2.2. Catchment Area. When the catchment, as seen from the "topo" sheet, is less than about half a square mile in area, a traverse should be made along the water shed with a chain and compass. Larger catchments can be read from the 1 inch = 1 mile topo maps of the Survey of India by marking the water shed in pencil and reading the included area by placing over it a piece of transparent square paper.

2.3. Cross Sections. As a rule, for a sizeable stream, three cross sections should be taken, namely, one at the selected site, one upstream and another downstream of the site. Approximate distances, upstream and downstream of the selected site of crossing, at which cross sections should be taken are as under;

Table 2.3.

Catchment Area.	Distance (u/s & d/s of the crossing) at which cross sections should be taken.
1. One square mile or less	500 ft
2. From 1 to 5 square miles	1000 ft
3. Over 5 square miles	$\frac{1}{4}$ mile to 1 mile.

The cross section at the proposed site of the crossing should show levels at close intervals and indicate outcrops of rocks, pools, etc. Often an existing road or a cart track crosses the stream at the site selected for the bridge. In such a case the cross section should not be taken along the centre line of the road or the track as that will not represent the natural shape and size of the channel. Instead, the cross section should be taken a short distance upstream or downstream of the selected site.

2.4. In the case of very small streams (catchments of 100 acres or less) one cross section may do but it should be carefully plotted so as to represent truly the normal size and shape of the channel on a straight reach.

2.5. All cross sections should be plotted to a suitable scale. If the horizontal and vertical scales are the same, the wetted perimeter can be measured directly from the cross section, otherwise it has to be calculated (see para 5.6).

2.6. The Maximum H. F. L. The maximum high flood level should be ascertained by intelligent local observation, supplemented by local enquiry, and marked on the cross sections.

2.7. Longitudinal Section. The longitudinal section should extend upstream and downstream of the proposed site for the distances indicated in the Table in para 2.3 and should show levels of the bed, the low water surface and the H. F. L.

2.8. Velocity Observation. Attempts should be made to observe the velocity during an actual flood and, if that flood is smaller than the maximum flood, the observed velocity should be suitably increased. The velocity thus obtained is a good check on the accuracy of that calculated theoretically.

2.9. Trial Pit Sections. Where the rock or some firm undisturbed soil stratum is not likely to be far below the alluvial bed of the stream, a trial pit should be dug down to such rock or firm soil. But, if there is no rock or undisturbed firm soil for a great depth below the stream bed level, then the trial pit may be taken down roughly 2 times the maximum depth of the (existing or anticipated) scour line. The location of each trial pit should be shown in the cross section of the proposed site. The trial pit section should be plotted to show the kind of soils passed through.

2.10. For very small culverts one trial pit will do. The results can be inset on the cross section.

2.11. In Conclusion. No satisfactory designing can be done unless the minimum essential design data listed above are collected and recorded.

These data can be collected with very little effort. Failure to collect these can only result in designs being based on guess work and such designs are likely to be either unnecessarily costly or to result in the failure of the structure when built.

ARTICLE 3

EMPIRICAL FORMULAE FOR PEAK RUN-OFF FROM CATCHMENT

3.1. Although records of rainfall exist to some extent, actual records of floods are seldom available in such sufficiency as to enable the engineer accurately to infer the worst flood conditions for which provision should be made in designing a bridge. Therefore recourse has to be taken to theoretical computations.

In this Article some of the most popular empirical formulae are mentioned.

3.2. Dickens Formula.

$$Q = CM^{\frac{3}{4}} \qquad \cdots \qquad \cdots \qquad (3.2)$$

Where, Q = the peak run-off in cusecs and M is the catchment area in square miles.

C = 800 – 1000 where the annual rainfall is 25–50 in.;
1000 – 1400 in Madhya Pradesh
1600 in Western Ghats.

3.3. Ryve's formula.

$$Q = CM^{\frac{2}{3}} \qquad \cdots \qquad \cdots \qquad (3.3)$$

This formula was devised for Madras.

Q = run-off in cusecs and M is the catchment area in sq. miles.

C = 450 for areas within 15 miles of the coast.
563 ,, between 15 and 100 miles of the coast.
673 for limited areas near the hills.

3.4. These empirical formulae involve only one factor viz., the area of the catchment and all the so many other factors that affect the run-off have to be taken care of in selecting an appropriate value of the coefficient. This is extreme simplification of the problem and cannot be expected to yield accurate results.

3.5. A correct value of C can only be derived for a given region from an extensive analytical study of the measured flood

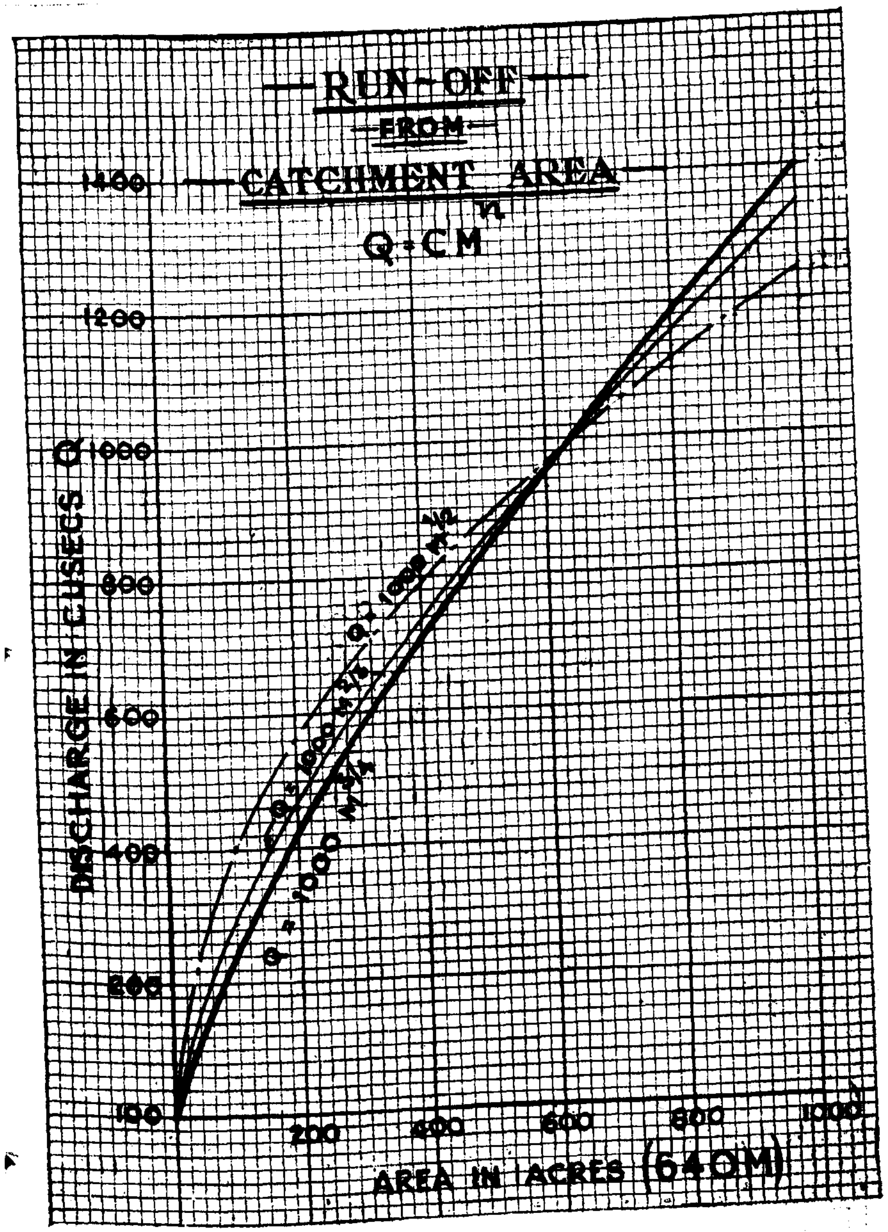


Fig. 3.6$_a$

discharges vis-a-vis catchment areas of streams in the region. Any value of C will be valid only for the region for which it has been determined in this way. Each basin has its own singularities affecting run-off. Since actual flood records are seldom available, the formulae leave much to the judgement of the engineer. Many other similar empirical formulae are in use but none of them encompasses all possible conditions of terrain and climate.

3.6. To assist routine computations Fig. 3.6 has been included in which curves have been plotted to represent the equation $Q=1000M^n$. Three curves, one for each of the common values of n, viz., $\frac{1}{2}$, $\frac{2}{3}$ and $\frac{3}{4}$, have been drawn.

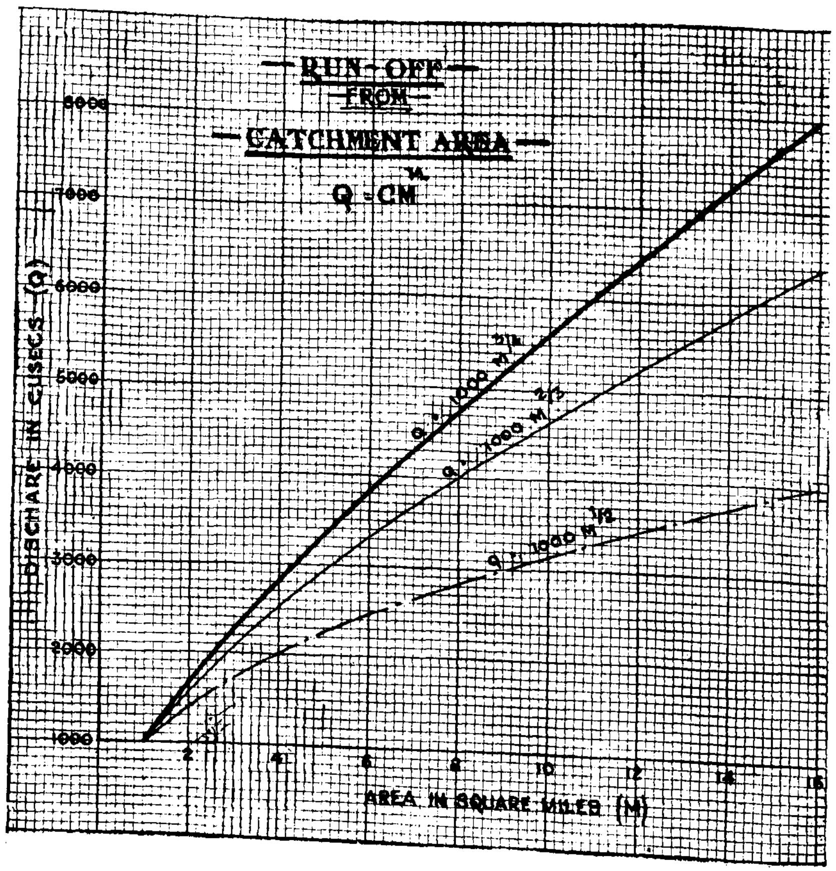


Fig. 3.6$_b$

3.7. American Formulae.

In America the McMath and Burkli-Zeigler formulas are popular. The former is :

$$Q=C.I_c.S^{0.2}.A^{0.8} \qquad \ldots \qquad \ldots \qquad (3.7)$$

where

$Q=$ the peak run-off in cusecs.

$S=$ fall in feet per 100 ft length of the main channel of flow.

$A=$ area of basin in acres.

$I_c=$ critical intensity of rainfall in inches per hour (see next Article).

$C=$ a coefficient.

$=0.10$ for large, weedy, flat areas ;

$=0.20$ for cultivated farms ;

$=0.30$ for thinly populated and unpaved suburbs, with gardens and lawns.

$=0.40$ for towns, not densely built upon.

ARTICLE 4

RATIONAL FORMULA FOR PEAK RUN-OFF FROM CATCHMENT

4.1. In recent years hydrological studies have been made and theories set forth which comprehend the effect of the characteristics of the catchment on run-off. Attempts also have been made to establish relationships between rainfall and run-off under various circumstances. Some elementary account of the rationale of these theories is given in the following paragraphs.

4.2. Main Factors. The size of the flood depends on the following major factors :

Rainfall

(1) Intensity
(2) Distribution in time and space
(3) Duration

Nature of the Catchment.

(4) Area
(5) Shape
(6) Slope
(7) Permeability of the soil and vegetable cover
(8) Initial state of wetness.

4.3. Relation between the Intensity and Duration of a Storm. Suppose in an individual storm, F inches of rain fall in T hours, then over the whole interval of time T, the mean intensity I will be $\frac{F}{T}$ inches per hour. Now, within the duration T imagine a smaller time interval t (Fig. 4.3). Since the intensity is not uniform throughout, the mean intensity reckoned over the time interval t (placed suitably within T) will be higher than the mean intensity I taken over the whole period.

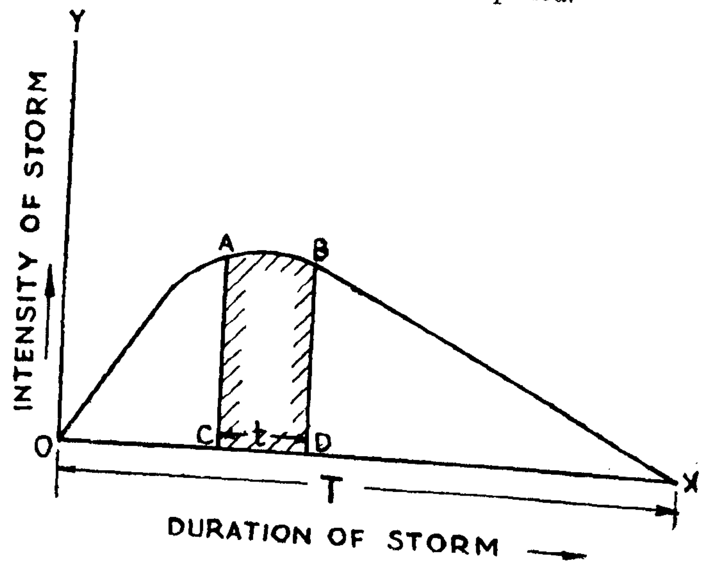


Fig. 4.3.

We also know that the mean intensity of a storm of shorter duration can be higher than that of a prolonged one.

In other words, the intensity of a storm is some inverse function of its duration. It has been reasonably well established that

$$\frac{i}{I} = \frac{T + c}{t + c}$$

where c is a constant.

Analysis of rainfall statistics has shown that for all but extreme cases, $c = 1$ [5]*, when time is measured in hours and precipitation in inches.

Thus,

$$\frac{i}{I} = \frac{T + 1}{t + 1}$$

or

$$i = I.\left(\frac{T + 1}{t + 1}\right) \quad (4.3_a)$$

Also,

$$i = \frac{F}{T}.\left(\frac{T + 1}{t + 1}\right) \quad (4.3_b)$$

Thus, if we know the total precipitation F and duration T of a storm we can estimate the intensity corresponding to t, which is a time interval within the duration of the storm.

4.4. For an appreciation of the physical significance of this relationship let us consider some typical cases.

First, take an intense but brief storm which drops (say) 1.5 inches of rain in 20 minutes. The average intensity comes to 4.5 inches per hour. For a short interval t of say 6 minutes within the duration of the storm the intensity can be as high as

$$i = \frac{F}{T}\left(\frac{T + 1}{t + 1}\right) = \frac{1.5}{0.33}\left(\frac{0.33 + 1}{0.1 + 1}\right) = 5.55 \text{ inches per hour.}$$

Storms of very short duration and 6-minute intensities within them (and in general all such high but momentary intensities of rainfall) have little significance in connection with the design of culverts except in built-up areas where the concentration time can be very short (see para 4.6) due to the rapidity of flow from pavements and roofs.

Next consider a region where storms are of medium size and duration. Suppose 5 inches of rain fall in 3 hours. The average intensity works out to 1.67 inches per hour. But in a time interval of one hour within the storm the intensity can be as much as

$$\frac{5}{3}\left(\frac{3 + 1}{1 + 1}\right) = 3.33 \text{ inches per hour.}$$

For the purposes of designing waterways of bridges such a storm is said to be equivalent of a "One hour rainfall of 3.33 inches".

Lastly, consider a very wet region of prolonged storms, where a storm drops, say, 7 inches of rain in 6 hours. In a

*Refers to the number of the book in the Bibliography given at the end of the Paper.

time interval of one hour within the storm the intensity can as be high as

$$\frac{7}{6} \times \left(\frac{6+1}{1+1}\right) = 4.08 \text{ inches per hour.}$$

Thus such a storm is equivalent of a "one hour storm of 4.08 inches".

4.5. "One-hour Rainfall" for a Region for Designing Water-ways of Bridges. Suppose we decide that a bridge should be designed for the peak run off resulting from the severest storm (in the region) that occurs once in 50 years or any other specified period. Let the total precipitation of that storm be F inches and duration T hours. Consider a time interval of one hour somewhere within the duration of the storm. The precipitation in that hour could be as high as :

$$\frac{F}{T}\left(\frac{T+1}{1+1}\right) \text{ or } \frac{1}{2} F\left(1+\frac{1}{T}\right) \text{ inches.}$$

Hence the design of the bridge will be based on a "one-hour rainfall of say I_0 inches", where

$$I_0 = \frac{F}{2}\left(1+\frac{1}{T}\right) \ldots\ldots\ldots\ldots\ldots \quad (4.5)$$

Suppose Fig 4.3 represents the severest storm experienced in a region. If t represents one hour, then the shaded area $ABCD$ will represent I_0.

It is convenient and common that the storm potential of a region for a given period of years should be characterised by specifying the "one hour rainfall" I_0 of the region for the purposes of designing the waterways of bridges in that region.

I_0 has to be determined from the F and T of the severest storm. That storm may not necessarily be the most prolonged storm. The correct procedure for finding I_0 is to take a number of really heavy and prolonged storms and work out I_0 from the F and T of each of them. The maximum of the values of I_0 thus found should be accepted as "the one hour rainfall" of the region for designing bridges.

The I_0 of a region does not have to be found for each design problem. It is a characteristic of the whole region and applies to a pretty vast area subject to the same weather conditions. I_0 of a region should be found once for all and should be known to the local engineers.

The Meteorological Department of the Government of India

has been approached to supply information* about the total precipitation F and the duration T of a number of heavy storms recorded at as many places in India as they can. When that information is available, it will be possible to analyse it and recommend values of I_0 for various parts of India for storms of different frequencies. Exceptional cases excluded, it will probably be sufficient in the meanwhile to take I_0 as 4 to 5 in. for areas of intense and prolonged rainfall and 2 to 3 in. for other areas in India for the purposes of designing minor bridges.

We start with I_0 and then modify it to suit the concentration time (see next para) of the catchment area in each specific case. This will now be explained.

4.6. Time of Concentration. (t_c). The time taken by the run-off from the farthest point on the periphery of the catchment (called the critical point) to reach the site of the culvert is called the "concentration time".

In considering the intensity of precipitation it was said that the shorter the duration considered the higher the intensity will be. Thus safety would seem to lie in designing for a high intensity corresponding to a very small interval of time. But this interval should not be shorter than the concentration time of the catchment under consideration, as otherwise the flow from distant parts of the catchment will not be able to reach the bridge in time to make its contribution in raising the peak discharge. Therefore, when we are examining a particular catchment, we need only consider the intensity corresponding to the duration equal to the concentration period (t_c) of the catchment.

4.7. Estimating the Concentration Time of a Catchment (t_c). The concentration time depends on (1) the distance from the critical point to the culvert; and (2) the average velocity of flow. The latter is governed by the slope and the roughness of the drainage channel and the depth of flow. Complicated formulae exist for deriving the time of concentration from the characteristics of the catchment. For our purposes, however, the following simple relationship [11] will do.

$$t_c = \left(11.9 \times \frac{L^3}{H}\right)^{0.385} \quad \ldots \quad (4.7)$$

where t_c = the concentration time in hours.

L = the distance from the critical point to the culvert in *miles*.

H = the fall in level from the critical point to the culvert in *ft*

* Since received and included in the Appendix.

L and H can be found from the survey plans of the catchment area and t_c calculated from Equation (4.7).

Plate I contains graphs from which t_c can be directly read for known values of L and H.

4.8. The Critical or Design Intensity. The critical intensity for a catchment is that maximum intensity which can occur in a time interval equal to the concentration time t_c of the catchment during the severest storm (in the region) of a given frequency. Call it I_c. Since each catchment has its own t_c, it will have its own I_c.

If we put $t = t_c$ in the basic equation (Equation 4.3_b) and write I_c for the resulting intensity, we get,

$$I_c = \frac{F}{T}\left(\frac{T+1}{t_c+1}\right) \quad \cdots \quad (4.8_a)$$

Combine this with Equation (4.5), we get

$$I_c = I_o\left(\frac{2}{t_c+1}\right) \quad \cdots \quad (4.8_b)$$

4.9. Calculation of Run-Off. When one inch of rain falls over one acre we get 3600 cu. ft. (precisely, $43560 \times \frac{1}{12} = 3630$ cu. ft.) of water. If this happens in one hour we get 3600 cu. ft. per hour or 1 cu. ft. per second. Hence :

> " A precipitation of one inch per hour over an area of one acre gives rise to a run-off of one cusec ".

It follows that a precipation of I_c inches per hour over an area of A acres, will give rise to a run-off

$$Q = A.I_c \text{ cusecs.}$$

To account for losses due to absorption etc. introduce a coefficient P. Then,

$$Q = P.A.I_c \quad \cdots \quad (4.9)$$

where Q = Max. run off in *cusecs*

A = Area of catchment in *acres*

I_c = Critical intensity of rainfall in **in. per hour.**

P = Percentage Coefficient of run-off for the catchment characteristics.

The principal factors governing P are : (i) porosity of the soil (ii) area, shape and size of the catchment (iii) vegetation cover, (iv) surface storage viz., existence of lakes and marshes (v) initial state of wetness of the soil. Catchments vary so much with regard to these characteristics that it is evidently impossible to do more than generalise on the values of P. Judgment and experience must be used in fixing P. Also see Table 4.9 for guidance.

TABLE 4.9

MAXIMUM VALUE OF P IN THE FORMULA $Q = P.A.I_c$.

Steep, bare rock. Also city pavements	...	0.90
Rock, steep but wooded ...	...	0.80
Plateaus, lightly covered	...	0.70
Clayey soils, stiff and bare	...	0.60
do lightly covered	...	0.50
Loam, lightly cultivated or covered	...	0.40
do largely cultivated	...	0.30
Sandy soil, light growth	...	0.20
Sandy soil, covered, heavy brush	...	0.10

4.10. Relation between Intensity and Spread of Storm. We have so far deliberately eschewed any mention of the spread of the storm. Rainfall recording stations are points in the space and therefore the intensities recorded there are point intensities. Imagine an area round a recording station. The intensity will be highest at the centre and will gradually diminish as we go farther away from the centre, till at the fringes of the area covered by the storm, intensity will be zero. The larger the area considered the smaller will be the mean intensity. It is therefore logical to say that the mean intensity is some inverse function of the size of the area.

If I is the maximum point intensity at the centre of the storm then the mean intensity reckoned over an area " a " is some fraction " f " of I. The fraction f depends on the area " a " and the relation is represented by the curve in Figure 4.10 which has been constructed from statistical analysis.[5]

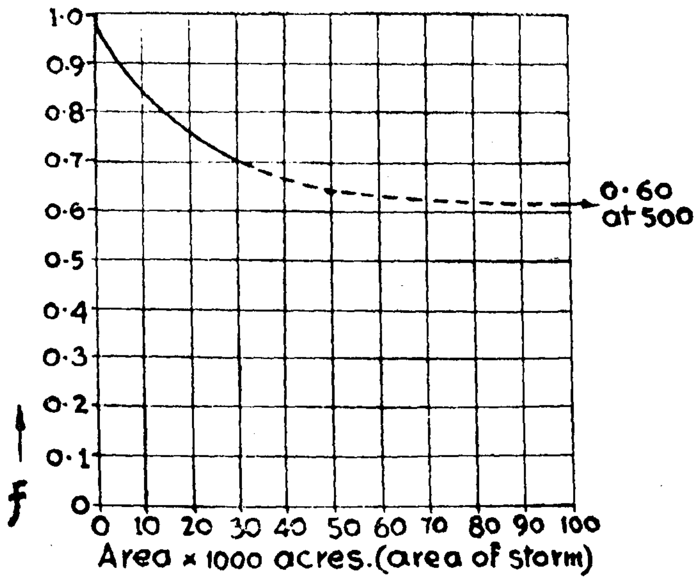


Fig. 4.10.

In hydrological theories it is assumed that the spread of the storm is equal to the area of the catchment. Therefore in Fig. 4.10 the area is taken to be the same as the area of the catchment. The effect of this assumption can lead to errors which, on analysis, have been found to be limited to about 12 per cent.[5]

4.11. The Final Run-off Formula. Introducing the factor f in the Equation 4.9 we get,

$$Q = P.f.A.I_o \ldots\ldots\ldots\ldots \ldots\ldots\ldots\ldots (4.11_a)$$

Also, combining with Equation (4.8_b)

$$Q = P.f.A.I_o.\left(\frac{2}{t_c+1}\right)$$

$$= A.I_o.\left(\frac{2.f.P}{t_c+1}\right)$$

$$= A.I_o.\lambda \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4.11_b)$$

In the last equation I_o measures the role played by the clouds of the region and λ that of the catchment in producing the peak run-off.

It should be clear from the foregoing paras that the components of λ are functions of A, L, and H of the catchment.

4.12. Resume of the Steps for Calculating the Run-off.

Step 1. Note down A in acres, L in miles, and H in feet from the survey maps of the area.

Step 2. Estimate the I_o for the region preferably from rainfall records: failing that from local knowledge. $I_o = \frac{F}{2}.(1+\frac{1}{T})$, where F is inches of rain dropped by the severest storm in T hours.

Step. 3. See **Plate I** and read values of t_c, P, and f for known values of L, H, and A.

Then calculate $\lambda = \frac{2fP}{t_c+1}$.

Step 4. Calculate $Q = A.I_o.\lambda$.

4.13. Example. Calculate the peak run-off for designing a bridge across a stream, given:

Catchment: $L = 3$ miles ; $H = 100$ ft ;
$A = 3.5$ sq. miles. Loamy soil largely cultivated.

Rainfall: The severest storm that is known to have occurred in 20 years dropped 5 inches of rain in 2.5 hours.

Solution. $I_o = 5 \left(\frac{2.5+1}{1+1}\right) = 3.5$ Inches/Hour

From **Plate I**,

$t_c = 1.6$ hours ; $f = 0.97$; $P = 0.30$

$$\therefore \lambda = \frac{2 \times 0.97 \times 0.30}{1.6+1} = 0.224$$

$$Q = (3.5 \times 640) \times 3.5 \times 0.22 = 1730 \text{ cusecs.}$$

4.14. Run-off Curves for Small Catchment Areas (Plate 2). Suppose we know the catchment area A in acres and the average slope S of the main drainage channel. If we assume that the length of the catchment is 3 times its width, then both L and H (as defined in para 4.7) can be expressed in terms of A and S and then t_c calculated from Equation (4.7).

Also for small areas, f may be taken equal to one. Then, vide para 4.11,

$$Q = P.\ I_0 A \left(\frac{2}{t_c+1}\right)$$

For $I_o = 1$ inch, this becomes,

$$Q = P.A\left(\frac{2}{t_c+1}\right) \quad \ldots \quad \ldots \quad (4.14)$$

Hence Q can be calculated for various values of P, A and S. This has been done and curves plotted in PLATE 2.

4.15. PLATE 2 can be used for small culverts with basins upto 3000 acres or about 5.0 square miles. The values of run-off read from Plate 2 are for "one hour rainfall", I_0, of **one inch** These values have to be multiplied by the I_o of the region. An example will illustrate the use of this Plate.

4.16. Example: The basin of a stream is loamy soil, largely cultivated, and the area of the catchment is 850 acres. The average slope of the stream is 5 per cent. Calculate the run-off. (I_0, the one hour rainfall of the region, is 2.5 inches).

Use Plate 2. For largely cultivated loamy soil, $P=0.30$ vide the Table inset in Plate 2.

Enter the diagram at $A=850$ acres: move vertically up to intersect the slope line of 5 per cent. Then, move horizontally to intersect the 00 line; join the intersection with $P=0.3$ and extend to the run off (q) scale and read

$$q = 350 \text{ cusecs.}$$

Multiply with I_o

$$Q = 350 \times 2.5 = 875 \text{ cusecs.}$$

4.17. In Conclusion.

The use of empirical formulae should be avoided. They are primitive and are safe only in the hands of the expert. The average designer who cannot rely so much on his intuition and judgment should go by the rational procedure outlined above.

The data required for the rational treatment, viz., A, L, and H can be easily read from the survey plans. As regards I_0 it should be realized that this does not have to be calculated for each design problem. This is the storm characteristic of the whole region, with pretty vast area, and should be known to the local engineers.

Complicated formulae, of which there is quite an abundance, have been purposely avoided in this Article. Indeed, for a terse treatment, the factors involved are so many and their interplay so complicated that recourse need be taken to such treatment only when very important structures are involved and accurate data can be collected. For small bridges the simple formulae given here should do.

ARTICLE 5

ESTIMATING FLOOD DISCHARGE FROM THE CONVEYANCE FACTOR AND SLOPE OF THE STREAM

5.1. In a stream with rigid boundaries (bed and banks) the shape and the size of the cross section is significantly the same during a flood as after its subsidence. If we carefully plot the H. F. L. and measure the bed slope it is simple to calculate the discharge.

5.2. But a stream flowing in alluvium, will have a larger cross-sectional area when in flood than that which may be surveyed and plotted after the flood has subsided. During the flood the velocity is high and therefore an alluvial stream scours its bed; but when the flood subsides, the velocity diminishes and the bed progressively silts up again. From this it follows that, before we start estimating the flood conveying capacity of the stream from the plotted cross section, we should ascertain the depth of scour and plot on the cross section the average scoured bed line that is likely to prevail during the high flood.

5.3. The best thing to do is to inspect the scour holes in the vicinty of the site; look at the size and the degree of incoherence of the grains of the bed material; have an idea of the probable velocity of flow during the flood; study the trial bore section; and then judge what should be taken as the probable average scoured bed line.

5.4. Calculation of Velocity.

Plot the probable scoured bed line. Measure the cross sectional area A in sq. ft. and the wetted perimeter P in ft. Then, calculate the hydraulic mean depth $R = \frac{A}{P}$ ft

Next, measure the bed slope S from the plotted longitudinal section of the Stream. Velocity can then be easily calculated from one of the many formulae. To mention one, viz., Manning's Formula :

$$V = \frac{1.4858}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots (5.4_a)$$

Where,

V = the Velocity in ft per sec. considered uniform throughout the cross section ;

R = the hydraulic mean depth ;

S = the energy slope which may be taken equal to the bed slope, measured over a reasonably long reach ;

n = the rugosity co-efficient.

For values of n see Table 5.4 below. Judgement and experience are necessary in selecting a proper value of n for a given stream.

TABLE 5.4

Rugosity Co-Efficient.

Values of " n " in the Formula $V = \frac{1.4858}{n} \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$

Surface	Perfect	Good	Fair	Bad
Natural streams				
(1) Clean, straight bank, full stage, no rifts or deep pools ...	0.025	0.0275	0.030	0.033
(2) Same as (1), but some weeds and stones ...	0.030	0.033	0.035	0.040
(3) Winding, some pools and shoals, clean ...	0.035	0.040	0.045	0.050
(4) Same as (3), lower stages, more ineffective slope and sections	0.040	0.045	0.050	0.055
(5) Same as (3), some weeds and stones ...	0.033	0.035	0.040	0.045
(6) Same as (4), stony sections ...	0.045	0.050	0.055	0.060
(7) Sluggish river reaches, rather weedy or with very deep pools ...	0.050	0.060	0.070	0.080
(8) Very weedy reaches ...	0.075	0.100	0.125	0.150

5.5. Calculation of Discharge.

$$Q = A.V.$$

$$= \frac{1.4858}{n} \cdot A.R^{\frac{2}{3}} \cdot S^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots (5.5)$$

$$= \lambda . S^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots (5.6)$$

where $\lambda = \frac{1.4858}{n} \cdot A.\ R^{\frac{2}{3}}$.

Now λ is a function of the size, shape, and roughness of the stream and is called its conveyance factor. Thus the discharge carrying capacity of a stream depends on its conveyance factor and slope.

5.6. When the cross section is not plotted to the natural scale (the same horizontally and vertically) the wetted perimeter cannot be scaled off directly from the section and has to be calculated.

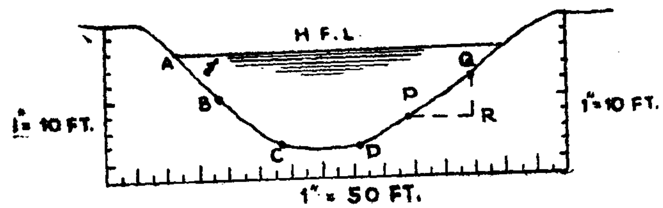


Fig. 5.6.

Divide up the wetted line into a convenient number of parts, AB, BC, CD etc. (Fig. 5.6). Consider one such part, say PQ. Let PR and QR be its horizontal and vertical projections. Then, $PQ = \sqrt{(PR^2 + QR^2)}$. Now, PR can be measured on the horizontal scale of the given cross-section and QR on the vertical. PQ can then be calculated. Similarly each of the other parts is calculated. Their sum gives the wetted perimeter.

5.7. If the shape of the cross section is irregular as happens when a stream rises above its banks and shallow over-flows are created (Fig. 5.7), it is necessary to subdivide the channel into two or three subsections. Then R and n are found for each subsection, and their velocities and discharges computed separately.

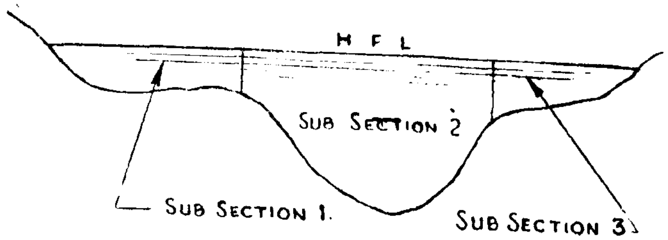


FIG. 5.7.

Where further elaboration is justified, corrections for velocity distribution, change of slope, etc. may be applied. Books on Hydraulics give standard methods for this.

5.8. Velocity Curves. To save time in computation, curves have been plotted in **Plate 3.** Given R, S, and n, velocity can be read from this Plate.

5.9. Better Measure than Calculate Velocity. It is preferable actually to observe the velocity during a high flood. When it is not possible to wait for the occurrence of a high flood, the velocity may be observed in a moderate flood and used as a check on the theoretical calculation of velocity. In making velocity observations the selected reach should be straight, uniform, and reasonably long.

5.10. The flood discharge should be calculated at each of the three cross sections, which, as already explained in para 2.3, should be plotted for all except very small structures. If the difference in the three discharges thus calculated is more than 10 per cent the discrepancy has to be investigated.

ARTICLE 6

ESTIMATING FLOOD DISCHARGE FROM FLOOD MARKS ON AN EXISTING STRUCTURE

6.1. Having collected the necessary information from inspection as mentioned in para 1.2 the discharge passed by an existing structure can be calculated by applying an appropriate formula. In Article 17, some formulae for calculating discharges from flood marks on existing bridges are discussed. Worked out examples have been included in Article 19.

6.2. Distinct water marks on bridge piers and other structures can be easily found immediately following the flood. Sometimes these marks can be identified years afterwards but it is advisable to survey them as soon after the flood as possible. Turbulence, standing wave, and splashing may have caused a spread in the flood marks but the belt of this spread is mostly narrow and a reasonably correct profile of the surface line can be traced on the sides of piers and faces of abutments.

6.3. This is perhaps the most reliable way of estimating a flood discharge, because in the formulae discussed in Article 17 the co-efficients involved have been accurately found by experiments.

ARTICLE 7

FIXING DESIGN DISCHARGE

7.1. The Recommended Rule. Flood discharges can be estimated in three different ways as explained in Articles 4 to 6. The values obtained should be compared. The highest of these values should be adopted as the design discharge Q, provided it does not exceed the next highest discharge by more than 50 per cent. If it does, restrict it to that limit.

7.2. Sound Economy. The designer is not expected to aim at designing a structure of such copious dimensions that it should pass a flood of any possible magnitude that can occur during the lifetime of the structure. Sound economy requires that the structure should be able easily to pass floods of a specified frequency (say once in 20 years for small bridges and 100 years for big ones) and that extraordinary and rarer floods should pass without doing excessive damage to the structure or the road.

7.3. The necessity for this elaborate procedure for fixing Q, arises for sizeable structures. As regards small culverts Q may be taken as the discharge determined from the run off formulae.

ARTICLE 8

ALLUVIAL STREAMS—LACEY'S EQUATIONS

8.1. The section of a stream, having rigid boundaries, is the same during the flood and after its subsidence. But this is not so in the case of streams flowing within, partially or wholly, erodible boundaries. In the latter case, a probable flood section has to be evolved from theoretical premises for the purposes of designing a bridge; it is seldom possible actually to measure the cross section during the high flood.

8.2. Wholly Erodible Section. Lacey's Theory. Streams flowing in alluvium are wide and shallow and meander a great deal. The surface width and the normal scoured depth of such streams have to be calculated theoretically from concepts which are not wholly rational. The theory that has gained wide popularity in India is "Lacey's Theory of Flow in Incoherent Alluvium". The salient points of that Theory, relevant to the present subject, are outlined here.

8.3. A stream, whose bed and banks are composed of loose granular material, that has been deposited by the stream and can be picked up and transported again by the current during flood, is said to flow through incoherent alluvium and may be briefly referred to as an alluvial stream. Such a stream tends to scour or silt up till it has acquired such a cross section and (more particularly) such a slope that the resulting velocity is "non-silting and non-scouring". When this happens the stream becomes stable and tends to maintain the acquired shape and size of its cross section and the acquired slope. It is then said "to have come to regime" and can be regarded as stable.

8.4. Lacey's Equations. When an alluvial stream carrying a known discharge Q has come to regime, it has a regime wetted perimeter P, a regime slope S, and a regime hydraulic mean depth R. In consequence, it will have a fixed area of cross section A and a fixed velocity V.

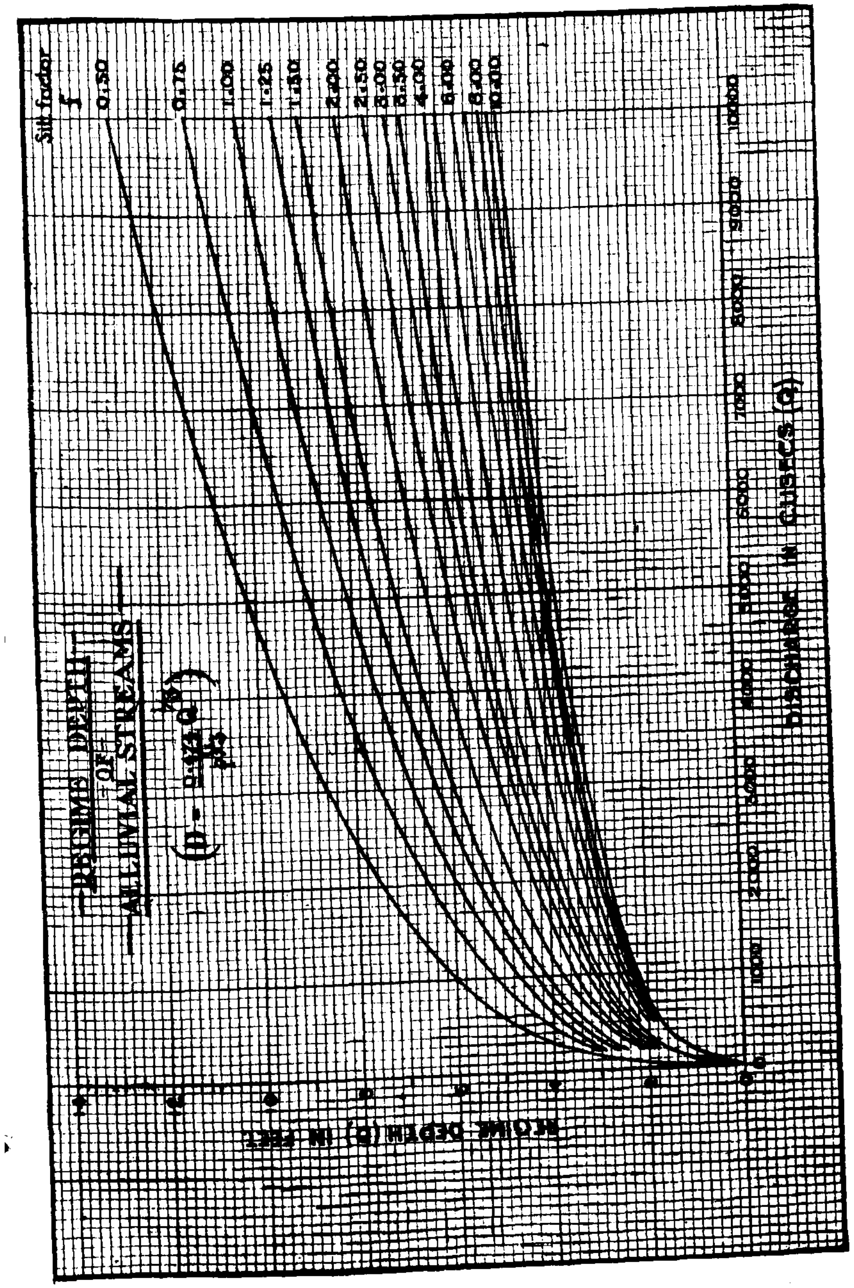
For these regime characteristics of an alluvial channel, Lacey suggests [18] the following relationships. It should be noted that the only independent entities involved in them are Q and f. The "f" is called **silt factor** and its value depends on the size and looseness of the grains of the alluvium. Table 8.4 gives values of "f".

(a) Regime Cross Section

$$P = \frac{8}{3} Q^{\frac{1}{2}} \qquad \ldots \quad (8.4_a)$$

$$R = \frac{0.473.Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \qquad \ldots \quad (8.4_b)$$

$$S = \frac{0.000.542.f^{\frac{5}{3}}}{Q^{\frac{1}{6}}} \qquad \ldots \quad (8.4_c)$$



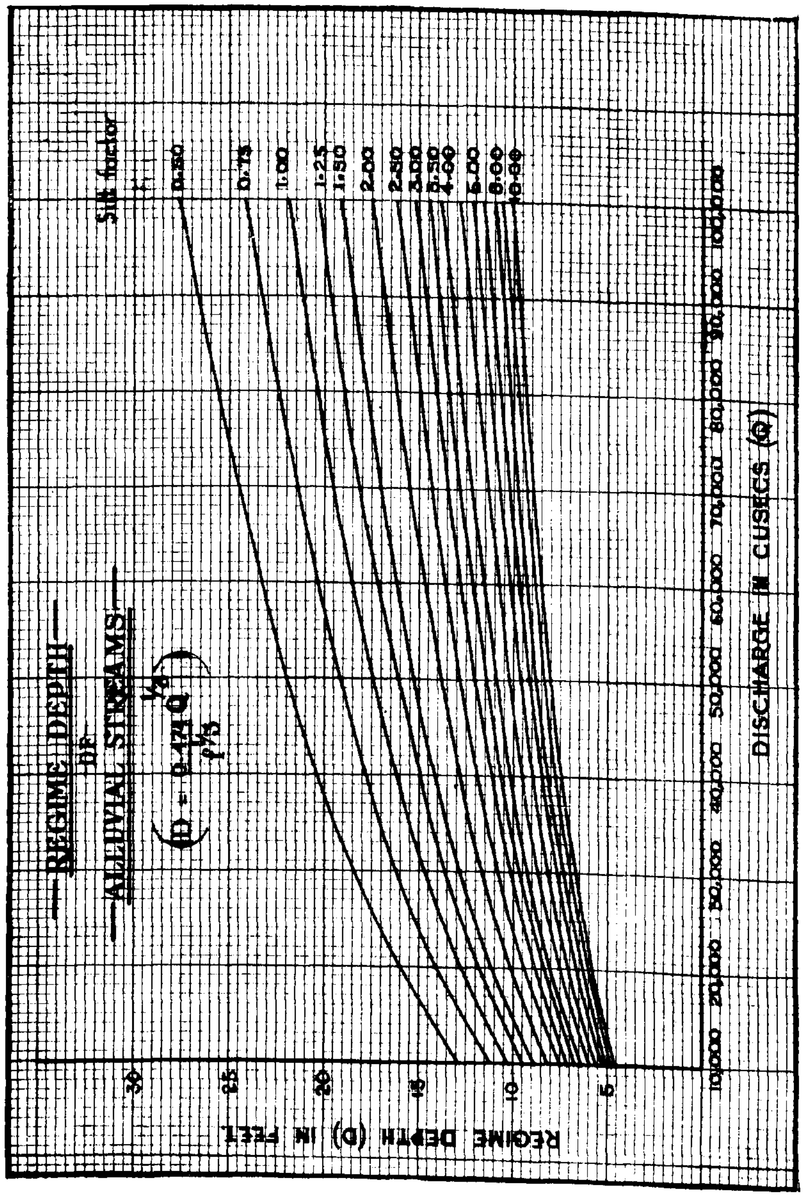


Fig. 8.6b

(b) Regime Velocity and Slope

$$V = 0.794\ Q^{\frac{1}{6}} \times f^{\frac{1}{3}} \quad \ldots \quad (8.4_d)$$

$$A = \frac{1.26.Q^{\frac{5}{6}}}{f^{\frac{1}{3}}} \quad \ldots \quad (8.4_e)$$

TABLE 8.4

SILT FACTOR "*f*" IN LACEY'S EQUATIONS[18]

		m Size of Grain in *mm*	*f* (Silt Factor)
Silt.			
	Fine	00.120	00.600
	Medium	00.233	00.850
	Standard	00.323	01.000
Sand.			
	Medium	00.505	01.250
	Coarse	00.725	01.500
Bajri.			
	Fine	00 988	01.750
	Medium	01.290	02.000
	Coarse	02.422	02.750
Gravel.			
	Medium	07.280	04.750
	Heavy	26.100	09.000
Boulders.			
	Small	50.100	12.000
	Medium	72.500	15.000
	Large	188.800	24.000

8.5. The Regime Width and Depth. Provided a stream is truly alluvial, it is destined to come to regime according to Lacey. It will then be stable and have a section and slope conforming to his equations. For wide alluvial streams the stable width W can be taken equal to the wetted perimeter P of Equation (8.4_a).

That is:

$$W = \frac{8}{3} \times Q^{\frac{1}{2}} \quad \ldots \quad (8.5_a)$$

Also the normal depth of scour D on a straight and unobstructed part of a wide stream may be taken as equal to the hydraulic mean radius R in Equation (8.4_b). Hence

$$D = \frac{0.473.Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \quad \cdots \quad (8.5_b)$$

8.6. Curves for D. In Fig. 8.6 curves have been plotted from Equation (8.5_b). Knowing Q and f for an alluvial stream, the regime depth D can be directly read off these curves.

ARTICLE 9

LINEAR WATERWAY OF BRIDGES

9.1. The General Rule for Alluvial Streams. The linear waterway of a bridge across a wholly alluvial stream should normally be kept equal to the width required for stability, viz. that given by Equation (8.5_a).

9.2. Unstable Meandering Streams. A large alluvial stream, meandering over a wide belt, may have several active channels separated by land or shallow sections of nearly stagnant water. Its actual (aggregate) width may be much in excess of the regime width required for stability. In bridging such a stream it is necessary to provide training works to contract the stream. The cost of the latter, both initial and recurring, has to be taken into account in fixing the linear waterway.

9.3. In the ultimate analysis it may be found, in some such cases, that it is cheaper to adopt a linear waterway for the bridge somewhat in excess of the regime width given by Equation (8.5_a). But, as far as possible, this should be avoided. When the adopted linear waterway exceeds the regime width it does not follow that the depth will become less than the regime depth D given by Equation (8.5_b). Hence such an increase in the length of the bridge does not lead to any countervailing saving in the depth of foundations. On the contrary, an excessive linear waterway can be detrimental in so far as it increases the action against the training works.

9.4. Contraction to be Avoided. The linear waterway of the bridge across an alluvial stream should not be less than the regime width of the stream. Any design, that envisages contraction of the stream beyond the regime width, necessarily has to provide for much deeper foundations. Much of the saving in cost expected from decreasing the length of the bridge may be eaten up by the concomitant increase in the depth of the substructure and the size of training works. Hence, except where the section of the stream is rigid, it is generally troublesome andalso futile from economy consideration to attempt contracting the waterway.

9.5. Streams Not Wholly Alluvial. When the banks of a stream are high, well defined, and rigid (rocky or of some other natural hard soil that cannot be affected by the prevailing current) but the bed is alluvial, the linear waterway of the bridge should be made equal to the actual surface width of the stream, measured from edge to edge of water along the H.F.L., on the plotted cross section.

Such streams are, later referred to as quasi-alluvial.

9.6. Streams with Rigid Boundaries. In wholly rigid streams the rule of para 9.5 applies, but some reduction in the linear waterway may, across some streams with moderate velocities, be possible and may be resorted to, if in the final analysis it leads to tangible saving in the cost of the bridge.

9.7. As regards streams which overflow their banks and create very wide surface widths with shallow side sections, judgement has to be used in fixing the linear waterway of the bridge. The bridge should span the active channel and detrimental afflux avoided. See, also, Article 20.

ARTICLE 10

THE NORMAL SCOUR DEPTH OF ALLUVIAL STREAMS.

10.1. What is the significance of the normal scour depth? If a constant discharge were passed through a straight stable reach of an alluvial stream for an indefinite time, the boundary of its cross section should ultimately become elliptical.

This will happen when regime conditions come to exist. The depth in the middle of the stream would then be the normal scour depth.

In nature, however, the flood discharge in a stream does not have indefinite duration. For this reason the shape of the flood section of any natural stream would be governed by the magnitude and duration of the flood discharge carried by it. Some observers have found that curves representing the natural stream sections during sustained floods have sharper curvature in the middle than that of an ellipse. In consequence, it is believed that Lacey's normal depth is an under-estimate when applied to natural streams subject to sustained floods. We will, however, apply Lacey's Equations, pending further research.

10.2. As discussed later in Article 14 the depth of foundations is fixed in relation to the maximum depth of scour, which in turn is inferred from the normal depth of scour. The normal depth of scour for alluvial streams is given by Equation (8.5_b) so long as the bridge does not contract the stream beyond the regime width W given by Equation (8.5_a)

10.3. If the linear waterway of the bridge is, for some special reason, kept less than the regime width of the stream, then the *normal* scour depth under the bridge will be greater than the regime depth of the stream (Fig. 10.3).

$$D' = D\left[\frac{W}{L}\right]^{0.61} \quad \cdots\cdots\cdots\cdots \textbf{(10.3)}$$

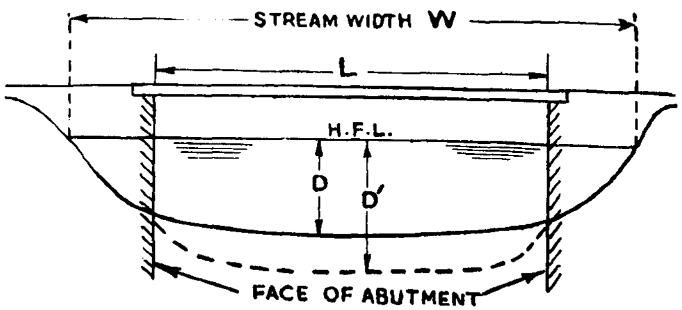
where, W = the regime width of the stream.

L = the designed waterway. When the bridge is assumed to cause contraction, L is less than W.

D = the normal scour depth when $L=W$.

D' = the normal scour depth under the bridge with L less than W.

This relationship is explained in Article 12.



This fig. represents a hypothetical condition i.e. Uniform scour. D is the normal scour depth corresponding to stream width W and D' normal scour depth corresponding to reduced waterway under the bridge.

Fig. 10.3.

ARTICLE 11

THE NORMAL SCOUR DEPTH OF QUASI ALLUVIAL STREAMS

11.1. Quasi-Alluvial Streams. Some streams are not wholly alluvial. A stream may flow between banks which are rigid in so far as they successfully resist erosion, but its bed may be composed of loose granular material which the current can pick up and transport. Such a stream may be called quasi-alluvial to distinguish it, on the one hand, from a stream with wholly rigid boundaries and, on the other, from a wholly alluvial stream. Since such a stream is not free to erode its banks and flatten out the boundaries of its cross section as a wholly alluvial stream does, it does not acquire the regime cross section which Lacey's Equations prescribe.

11.2. It is not essential that the banks should be of rock to be inerodible. Natural mixtures of sand and clay may, under the influence of elements, produce material hard enough to defy erosion by the prevailing velocity in the stream.

When we come across a stream section, the natural width of which is nowhere near that prescribed by Lacey's Theory, we should expect to find that the banks, even though not rocky, are not friable enough to be treated as incoherent alluvium for the application of Lacey's Theory. Such cases have, therefore, got to be discriminated from the wholly alluvial streams and treated on a different footing.

11.3. In any such case the width W of the section, being fixed between the rigid banks, can be measured. But the normal scour depth D corresponding to the design discharge Q has to be estimated theoretically as it cannot be measured during the currency of a high flood.

11.4. When the Stream Width is Large Compared to Depth. In Article 5, for calculating the discharge of the stream from its plotted cross section, we drew the probable scoured bed line (para 5.3). When the stream scours down to that line it should be capable of passing the discharge calculated there, say q cusecs. But the discharge adopted for design, Q, may be anything upto 50 per cent more than q (See para 7.1). Therefore, the scour bed line will have to be lowered further. Suppose the normal scour depth for Q is D and that for q is d, then,

$$D = d \times \left(\frac{Q}{q}\right)^{\frac{3}{5}} \qquad \textbf{(11.4)}$$

Since d is known, D can be calculated. This relationship depends on the assumption that the width of the stream is large as compared with its depth, and therefore, the wetted perimeter is approximately equal to the width and is not materially affected by variations in depth. It also assumes that the slope remains unaltered.

$$Q = \text{Area} \times \text{Velocity}.$$
$$= (R.P)\,(C.R^{\frac{2}{3}}\,S^{\frac{1}{2}}$$
$$= K.R^{\frac{5}{3}}$$

where K is a constant.

Hence, R varies as $Q^{\frac{3}{5}}$. Since in such streams R is very nearly equal to the depth, therefore, D varies as $Q^{\frac{3}{5}}$. Hence the Equation (11.4).

From the above relationship it follows that if Q is 150 per cent of q, D will equal to 1.27 per cent of d.

11.5. Alternatively the normal depth of scour of wide streams may be calculated as under. If the width of the stream is large as compared with its depth, we may write W for P and D for R.

$$Q = \text{Area} \times \text{Velocity}.$$
$$= (P.R) \times V = (W.D).V.$$
$$\text{i.e., } D = \frac{Q}{W.V}.$$

Now, W is the known fixed width of the stream. If the velocity V has actually been observed (para 5.9) then D can be calculated from the above equation.

11.6. Suppose the velocity has not been actually measured during a flood, but the slope S is known.

$$Q = \text{Area} \times \text{Velocity}.$$
$$= (R \times P) \times \left(\frac{1.4858}{n}\right) \quad R^{\frac{2}{3}} S^{\frac{1}{2}}$$
$$= \frac{1.4858}{n} \times W \times S^{\frac{1}{2}} \times D^{\frac{5}{3}} \qquad \textbf{(11.6)}$$

Knowing Q, W and S, we can calculate D from this equation.

For quickness, Velocity Curves in Plate 3 can be used. Assume a value of R and fix a suitable value of the rugosity

coefficient n appropriate for the stream. Corresponding to these values and the known slope, read the velocity from Plate 3. Now calculate the discharge ($= V \times R \times W$). If this equals the design discharge Q, the assumed value of R is correct. Otherwise assume another value of R and repeat. When the correct value of R has been found, take D equal to R. (See the worked out Example in para 18.8).

11.7. Another Formula. The procedure described above can be applied if we know either the slope of the stream or the actual observed velocity. If we do not known either of these, we can apply the following procedure for approximate calculation of the normal scour depth.

Suppose the wetted perimeter of the stream is P and its hydraulic mean depth R. If Q is its discharge, then,

$$Q = \text{Area} \times \text{Velocity}$$
$$= (P.R.)\,(C.\ R^{\frac{2}{3}}.S^{\frac{1}{2}}) \quad \ldots \quad \text{(i)}$$

Now if this stream, carrying the discharge Q, had been wholly alluvial, with a wetted perimeter P_1 and Hydraulic mean depth R_1 for regime conditions, then,

$$Q = (P_1.R_1)\,(C.R^{\frac{2}{3}} S^{\frac{1}{2}} \quad \ldots \quad \text{(ii)}$$

Also for a wholly alluvial stream Lacey's Theory would give :

$$P_1 = 2.67.Q^{\frac{1}{2}} \quad \ldots \quad \text{(iii)}$$

$$\text{and } R_1 = \frac{047.4Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \quad \ldots \quad \text{(iv)}$$

Equating values of Q in (i) and (ii), and rearranging we get :

$$\frac{R}{R_1} = \left(\frac{P_1}{P}\right)^{\frac{3}{5}} \quad \ldots \quad \text{(v)}$$

Now substituting values of P_1 and R_1 from equations (iii) and (iv) in (v), we get

$$R = \frac{0.86.Q^{0.63}}{f^{0.33} \times P^{0.60}} \quad \ldots \quad \text{(vi)}$$

If the width W of the stream is large compared with its depth D, then we can write W for P and D for R in equation (vi). This will give :

$$D = \frac{0.86.Q^{0.63}}{f^{0.33} \times W^{0.60}} \quad \ldots \quad \textbf{(11.7)}$$

Thus, if the design discharge Q, the natural width W, and the silt factor f are known, the normal scour depth D can be calculated from Equtaion (11.7).

The above reasoning assumes that the slope at the section in the actual case under consideration is the same as the slope of the hypothetical (Lacey's) regime section, carrying the same discharge. This is not improbable where the stream is old and its bed material is really incoherent alluvium. But if there is any doubt about this, the actual slope must be measured and the procedure given in para 11.6 applied.

11.8. When the Stream is Not Very Wide. If the width of the stream is not very large as compared with its depth, then the methods given above will not give accurate enough results. In such a case draw the probable scoured bed line on the plotted cross section, measure the area and the wetted perimeter and calculate R. Corresponding to this value of R and the known values of S and n, read velocity from **Plate 8**. If the product of this velocity and the area equals the design discharge, the assumed scoured bed line is correct. Otherwise, assume another line and repeat. Then measure D.

11.9. Effect of Contraction on Normal Scour Depth. If, for some special reason, the linear waterway L of a bridge across a quasi alluvial stream is kept less than the natural unobstructed width W of the stream, (Fig. 10.3), then the normal scour depth under the bridge D' will be greater than the depth D ascertained above for the unobstructed stream. The following relationship will apply.

$$D' = D\left(\frac{W}{L}\right)^{0.61} \quad \ldots \quad (14.9)$$

For proof see Article 12.

ARTICLE 12

THE EFFECT OF CONTRACTION ON THE NORMAL SCOUR DEPTH

12.1. In Fig. 12.1, W is the width, D the normal scour depth, and u the velocity of the stream in an unobstructed reach upstream of the bridge.

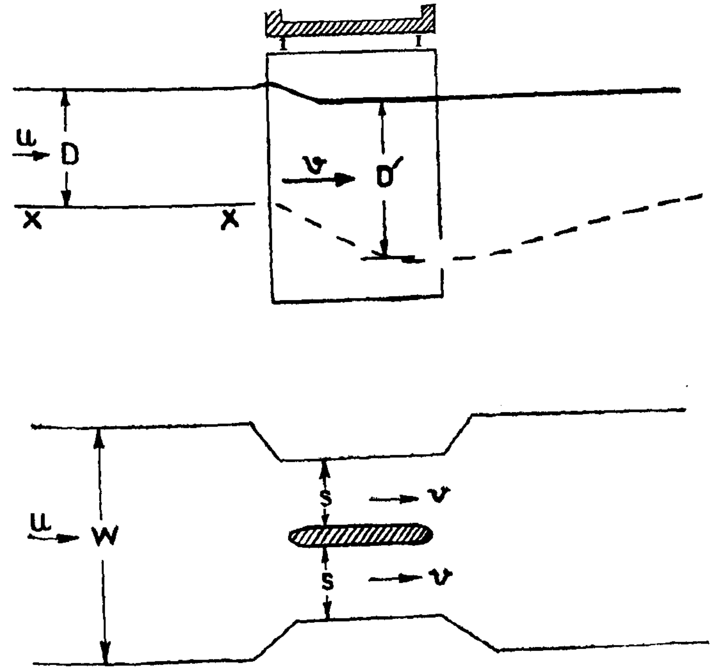


Fig. 12.1.

If XX is the normal scour bed line and further scouring has ceased, then according to Kennedy's Silt theory :

$$u = m \times (D)^{0.64} \quad \ldots\ldots \text{(i)}$$

Since the vent-area of the bridge is smaller than that of the stream, therefore, when the water arrives at the bridge the initial velocity V through the bridge will be greater than u. At that stage,

$$V = u.\ \frac{W}{L}$$

The initial high velocity V under the bridge is momentary. It starts further scouring and, in the process, itself begins to decrease (because further scouring means more area of flow through the bridge) till equilibrium is reached. If in that state of equilibrium, the depth under the bridge is D and the final reduced velocity v, then by Kennedy again,

$$v = m\ (D')^{0.64} \quad \ldots\ldots \text{(ii)}$$

From equations (i) and (ii)

$$\frac{v}{u} = \left(\frac{D'}{D}\right)^{0.64} \qquad \text{......(iii)}$$

Now, since the discharge at all sections is the same, therefore,

$$u.W.D = v.L.D'.$$

where L is the linear waterway through the bridge.

$$\text{i.e., } v = \left(\frac{W.D.}{L.D'}\right) \times u \qquad \text{...... (iv)}$$

substituting for v, from (iv) in (iii), we get:

$$D' = D\left(\frac{W}{L}\right)^{0.61} \qquad \text{......(12.1)}$$

Knowing D, W, and L we can calculate the increased depth of scour D' under the bridge from the above equation.

12.2. In this discussion it is tacitly assumed that scour develops uniformly and the bed line is lowered across the whole section of the bridge in a smooth curve. When that happens the depth D' will be the Normal scour depth. In natural streams, however, scour is not uniform. At some points there is deeper scour than that at others. An expression for the deepest scour in such cases will be developed later (See para 13.5).

ARTICLE 13

MAXIMUM SCOUR DEPTH

13.1. In considering bed scour we are concerned with alluvial and quasi-alluvial streams only and not with streams which have rigid beds.

13.2. In natural streams the scouring action of the current is not uniform all along the bed width. It is not so even in straight reaches. Particularly at the bends as also round obstructions to the flow, e.g. the piers of a bridge, there is deeper scour than normal. In the following paragraphs rules for calculating the maximum scour depth are given. It will be seen that the maximum scour depth is taken as a multiple of the normal scour depth according to the circumstances of the case.

13.3. In order to estimate the maximum scour depth, it is necessary first to calculate the normal scour depth. The latter has already been discussed in detail. To summarise what has

been said earlier, the **normal scour depth** will be calculated as under:

(i) Alluvial Streams. Provided the linear waterway of the bridge is not less than the regime width of the stream, the normal scour depth D is the regime depth as calculated from Equation (8.5_b).

(ii) Streams with Rigid Banks but Erodible Bed. Provided the linear waterway of the bridge is not less than the natural unobstructed surface width of the stream, the normal scour depth D is calculated as explained in Article 11.

(iii) Modification of Normal Scour Depth. When the bridge causes contraction, i.e., L is less than W, both foralluvial and quasi-alluvial streams, the modified normal scour depth D' is given by:

$$D' = D\left(\frac{W}{L}\right)^{0.61}$$

where, L = linear waterway of the bridge.

W = regime width in the case of alluvial streams and unobstructed natural width in the case of quasi alluvial streams.

13.4. Rules for Finding Maximum Scour Depth. The rules for calculating the maximum scour depth from the normal scour depth are:

Rule 1: For average conditions on a straight reach of the stream and when the bridge is a single span structure, i.e., it has no piers obstructing the flow, the maximum scour depth should be taken as 1.5 times the normal scour depth, modified for the effect of contraction where necessary.

Rule 2: For bad sites on curves or where diagonal currents exist or the bridge is a multi-span structure, the maximum scour depth should be taken as 2 times the normal scour depth, modified for the effect of contraction where necessary.

Rule 3: For bridges causing contraction, the maximum scour depth obtained by Rule 1 or 2, should be compared with that given by the following equation and the greater of the two values adopted.

$$D_m = D\left(\frac{W}{L}\right)^{1.56}$$

where D_m is the maximum scour depth, and

D, L, W have the same significance as above. An explanation of this last rule will now follow.

13.5. Explanation of the Formula.

$$D_m = D\left(\frac{W}{L}\right)^{1.56}$$

we have to revert to the discussion in Article 12. There we assumed that scouring under the bridge was uniform and worked out an expression for the normal scour depth under the bridge. In natural streams scouring is not uniform and deeper scour develops at some points than at others (Fig. 13.5). To work out what the maximum scour will be under such conditions we assume that, as scour develops, the velocity through deepened parts, P and Q in Fig. 13.5, does not decrease. More water can rush into portions of deeper scour from the sides and the initial velocity through the bridge viz., V is maintained in those portions.

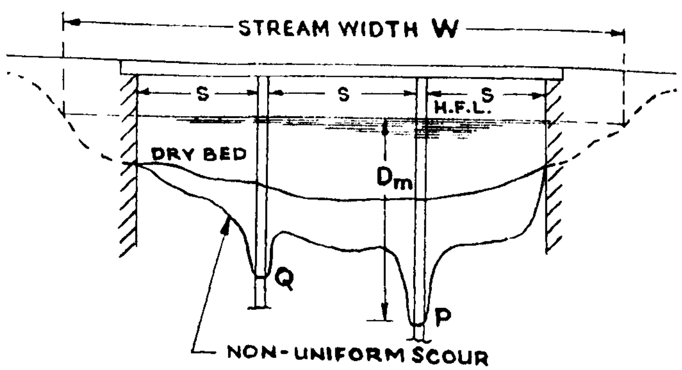


Fig. 13.5.

$$V = \frac{W}{L} \cdot u \qquad \ldots \quad \text{(i)}$$

When the maximum scour depth D_m has developed there will be equilibrium between V and D_m.

Hence, by Kennedy,

$$V = m\,(D_m)^{0.64} \qquad \ldots \quad \text{(ii)}$$

Also, as in Article (11)

$$u = m(D)^{0.64} \qquad \ldots \quad \text{(iii)}$$

From (ii) & (iii)

$$\frac{V}{u} = \left(\frac{D_m}{D}\right)^{0.64} \qquad \ldots \quad \text{(iv)}$$

Substituting for V from (i) in (iv) and rearranging

$$D_m = D\left(\frac{W}{L}\right)^{1.56} \qquad \text{(13.5)}$$

13.6. It is easy to prove that, where Rule 1 applies, the maximum scour depth given by it governs so long as the linear waterway of the bridge L is not less than 65 per cent of W. Beyond that limit, the maximum scour depth given by Equation (13.5) is greater.

But where Rule 2 applies, the maximum scour depth, given by it, will govern where L is anything down to 47 per cent of W. Since contractions below this limit are not resorted to in practice, checking by Rule 3 becomes needless.

Examples at the end of this Article will make this clear.

13.7. The finally adopted value of D_m must not be less than the depth (below H.F.L.) of the deepest scour hole that may be found by inspection to exist at or near the site of the bridge.

The following examples will illustrate the application of the rules in para 13.4 above.

13.8. Example I. A bridge is proposed across an alluvial stream (f = 1.2) carrying a discharge of 20,000 cusecs. Calculate the depth of maximum scour when the bridge consists of :

(a) 3 spans of 50, 75, & 50 ft ;

(b) 5 spans of 50 ft each ;

(c) 4 spans of 100 ft each.

Regime surface width of the stream

$$W = 2.67\, Q^{\frac{1}{2}} = 2.67 \times (20{,}000)^{\frac{1}{2}} = 380 \text{ ft}$$

Regime depth

$$D = 0.473\frac{Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} = \frac{0.473 \times (20{,}000)^{\frac{1}{3}}}{(1.2)^{\frac{1}{3}}} = 12.1 \text{ ft}$$

Case (a). Rule 2 applies, since the bridge has piers.

$D_m = 2 \times D' = 2 \times D(W/L)^{0.61}$

$= 2 \times 12.1\ (380/175)^{0.61} = 38.8$ ft

For check, calculate D_m by Equation (13.5)

$D_m = D(W/L)^{1.56} = 12.1 \times (380/175)^{1.56} = 40.6$ ft

Adopt 40.6 ft say 41 ft

[Note. In this case Equation (13.5) governs because the contraction exceeds 47%. Compare para 13.6.]

Case (b). Rule 2 applies.

$D_m = 2D' = 2D(W/L)^{0.61}$

$= 2 \times 12.1\ (380/250)^{0.61} = 31.2$ ft

Check by Equation (13.5),

$D_m = D\ (W/L)^{1.56} = 12.1\ (380/250)^{1.56} = 23.3.$

Adopt 31.2 say 31 ft

Case (c). Rule 2 applies.

$D_m = 2\ D = 2 \times 12.1 = 24.2$ ft

Equation (13.5) will not apply as $L = W$.

Adopt 24.2 say 24 ft

13.9. Example 2. A stream with rocky banks has a width of 300 ft. Its bed is alluvial (f = 1.2) and discharge 20,000 cusecs. Calculate the maximum scour depth under a bridge (a) a single arch span of 200 ft (b) three spans of total linear waterway 200 ft.

By Equation (11.7):

$D = 0.86.\ Q^{0.63}/(f^{0.33} \times W^{0.60})$

$= 0.86 \times (20{,}000)^{0.63}/(1.2^{0.33} \times 300^{0.6})$

$= 13.5$ ft

Case (a). By Rule 1:

$D_m = 1.5\ D' = 1.5\ D(W/L)^{0.61}$

$= 1.5 \times 13.5(300/200)^{0.61} = 26\ ft$

Check by Equation (13.5),

$D_m = D\ (W/L)^{1.56} = 13.5 \times (300/200)^{1.56}$

$= 25.5\ ft$

Adopt 26 ft

[Note, when contraction is 67%, Rule 1 and Equation (13.5) give equal values of D_m]

Case (b). By Rule 2:

$D_m = 2 \times D' = 2 \times D \times (W/L)^{0.61}$

$= 2 \times 13.5 \times (300/200)^{0.61} = 34.5$ ft

Check by Equation (13.5),

$D_m = D(W/L)^{1.56} = 13.5 \times (300/200)^{1.56}$

$= 25.4$ ft

Adopt 34.5 ft

Note: In this example the data do not mention the velocity or the slope of the stream, yet Equation (11.7) enables us to find the normal depth D of the unobstructed stream.

ARTICLE 14

DEPTH OF BRIDGE FOUNDATIONS (D_f)

14.1. The following rules have been based on the IRC Code of Practice for Road Bridges:

RULE (1) **Erodible Beds.** The foundations shall be taken down to a depth below the maximum high flood level one-third greater than the calculated depth of maximum scour subject to a minimum depth below the scour line of *six feet for arched bridges, and four feet for other bridges.*

RULE (2) **Hard Beds.** When a substantial stratum of solid rock or other material not erodible at the calculated maximum velocity is encountered at a level higher than or a little below that given by Rule (1) above, the foundations shall be securely anchored into that material. (This means about one foot or so into rock and about two feet or so into other hard material).

RULE (3) **All Beds.** The pressure on the foundation material must be well within the bearing capacity of the material.

These rules enable us to fix the level of the foundations of abutments and piers.

14.2. The rules mentioned in the last para apply when no bed floor is provided under the structure and the stream is free to scour as it may. The structure is then said to have deep foundations. It is sound to design deep foundations as far as circumstances permit.

14.3. In the case of small culverts, however, bed floors may be provided. In this connection the following is considered sound practice for culverts on erodible soil:

> Keep the top of the floor about a foot below the bed level. Take the foundations of the abutment 4 ft below the top of the floor. Provide an upstream curtain wall 3 to 5 ft deep and a downstream curtain wall 5 to 8 ft deep from the top of the floor. The precise depth of the curtain walls, within the stated limits, will depend on the velocity of flow through the structure and erodibility of the bed material.

This rule may be followed when it is intended to avoid calculation of maximum scour depths for want of sufficient site data.

ARTICLE 15

LENGTH OF CLEAR SPAN AND NUMBER OF SPANS

15.1. As a rule the number of spans should be as small as possible, since piers obstruct flow. Particularly in mountainous regions, where torrential velocities prevail, it is better to span from bank to bank using no piers if possible.

15.2. Considering only the variable items, the cost of superstructure increases and that of the substructure decreases with an increase in the span length. The most economical span length is that, which satisfies the equation:

$$\left.\begin{array}{l}\textbf{Cost of the}\\ \textbf{Superstructure}\end{array}\right\} = \left\{\begin{array}{l}\textbf{The Cost of the}\\ \textbf{Substructure}\end{array}\right.$$

15.3. Where the bridge is big, and difficult conditions are anticipated in the sinking of foundations, accurate and detailed examination of the economic aspect is essential and any deviation from the ideal case have to be justified strongly.

15.4. Length of Span. In small structures, where open foundations can be laid and solid abutments and piers raised on them, it has been anaylsed that the following approximate relationships give economical designs.

For masonry arched bridges:

$$S = 2H \qquad \ldots \qquad \ldots \qquad (15.4_a)$$

For R. C. C. Slab bridges:

$$S = 1.5H \qquad \ldots \qquad \ldots \qquad (15.4_b)$$

Where S = clear span length in ft

H = total height of abutment or pier from the bottom of its foundation to its top. For arched bridges it is measured from foundation to the intrados of the key stone.

15.5. We have earlier found the depth of foundations, D_f. We should now fix the vertical clearance, which, added to D_f, will give H. For vertical clearance the following is quoted from the I. R. C. Bridge Code (Clause 105).

Discharge	Vertical Clearance
Below 10 cusecs	6 inches
10–100 ,,	1.0 ft
100–1000 ,,	2.0 ft
1000–10,000 ,,	3.0 ft

In designing culverts for roads across flat regions where streams are wide and shallow (mostly undefined dips in the ground surface), and in consequence the natural velocities of flow are very low, the provision of clearance serves no purpose. Indeed it is proper to design such culverts on the assumption that the water at the inlet end will pond up and submerge the inlet to a predetermined extent. This will be discussed later in detail (Articles 21 and 22).

15.6. The Number of Spans. If the required linear waterway L is less than the economical span length it has to be provided, as it is, in one single span.

15.7. When L is more than the economical span length S, the number of spans required (N) is tentatively found from the following relation

$$L = NS.$$

15.8. Since N must be a whole number (preferably odd) S has to be modified suitably. In doing so it is permissible to adopt varying span lengths in one structure to keep as close as possible to the requirements of economy and to cause the least obstruction to the flow.

15.9. It is assumed that each bridge pier causes contraction in the filaments of water equal to half its thickness on each side (Fig. 15.9). For this reason the finally selected number and lengths of spans should be such that the sum of all clear span lengths should be equal to the required linear waterway plus the sum of the thicknesses of all piers.

Referring to Fig. 15.9,

$$\Sigma S = \Sigma l + \Sigma t \quad \ldots \quad (15.8_a)$$

Also, $$B = \Sigma l + 2\Sigma t \quad \ldots \quad (15.8_b)$$

Here Σl is the effective linear waterway required according to the design, ΣS = sum of all clear spans, Σt = sum of all pier thicknesses and B is the length of the bridge from face to face of abutments (See Fig 15.9).

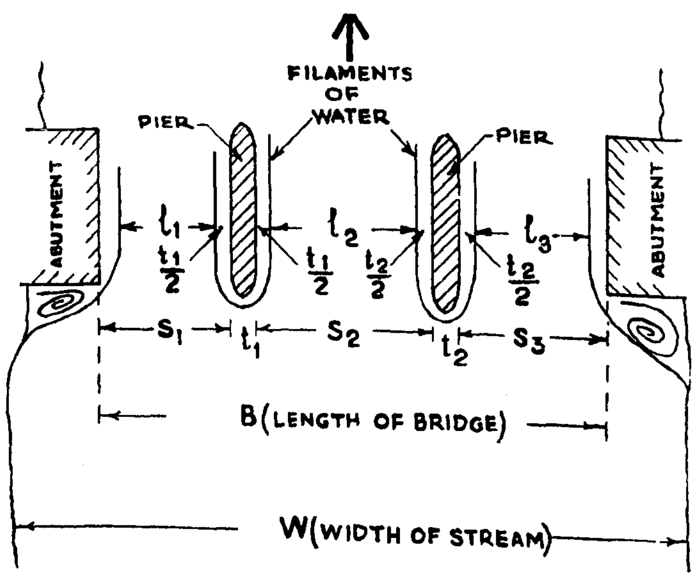


Fig. 15.9

ARTICLE 16

STRUCTURAL DETAILS OF MINOR BRIDGES AND CULVERTS

16.1. Abutment and Pier Sections. For masonry arched bridges abutment and pier sections should be taken from **Plate 4.**

As regards other types of small bridges, the sections in Fig. 16.1 may be adopted in simple cases not meriting elaborate calculations.

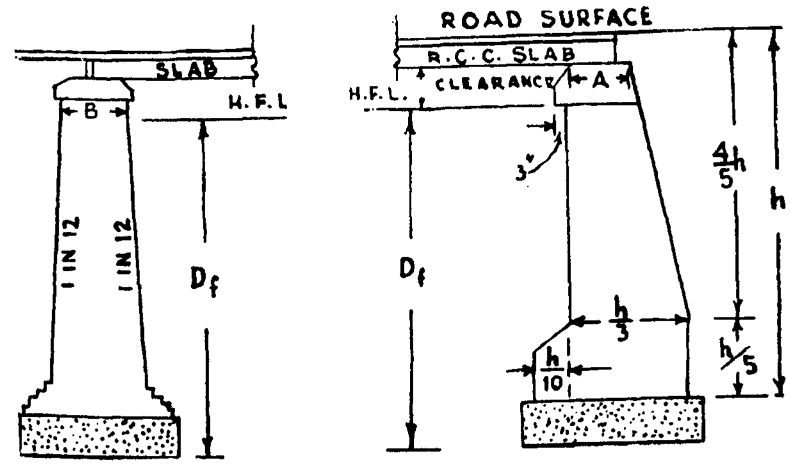


Fig. 16.1

The base widths of the abutment and the pier depend on the bearing power of the soil. The pressure at the toe of the abutment should be worked out to ensure that the soil is not over-stressed.

As regards top width " A " of the abutment and " B " of the pier, see Table 16.1 below.

TABLE 16.1

Top Widths of Abutments & Piers for Slab Bridges [14]

CLEAR SPAN	CEMENT MASONRY		LIME MASONRY	
	A of Abutment	B of pier	A of abutment	B of pier
Ft	Ins	Ins	Ins	Ins
10	15	18	18	18
20	18	22	24	24
30	22	26	27	30
40	26	30	32	36
50	30	35	37	42

16.2. Wing Wall and Return Walls. Long and hefty wing walls should be avoided. Where the foundations of the wing walls can be stepped up, having regard to the soil profile, this should be done for the sake of economy. Quite often short return walls meet the requirements of the case and should be adopted.

16.3. Unreinforced Masonry Arches. As far as possible unreinforced masonry arches should be used in preferences to R. C. C. slabs. The idea is to conserve steel (which is in short supply) for larger structures. The unreinforced masonry arches may be in stone, P. C. C. precast blocks, or bricks. **Plate 4** gives complete details of design [17]. When a masonry arch is used the sections of abutments and piers should also be taken from this **Plate.**

16.4. R. C. C. Slabs. If a R. C. C. slab has to be used, its dimensions should conform to those given in **Plate 5.**

16.5. Width of Roadway. The length of a culvert (along the direction of flow) should be such that the distance between the outer faces of the parapets will equal the full designed width of the formation of the road. Any proposed widening of the road formation in the near future should also be taken into account in fixing the width of the culvert.

In small bridges, the length (parallel to the flow of the stream) should be sufficient to give a minimum clear roadway of 24 ft between the inner faces of the kerbs or wheel guards. Extra provision should be made for foot paths etc., if any are required.

ARTICLE 17

ELEMENTS OF THE HYDRAULICS OF FLOW THROUGH BRIDGES

17.1. The formulae for discharges passing over broad crested weirs and drowned orifices have been developed **ab initio** in this Section. These formulae are very useful for computing flood discharges from the flood marks left on the piers and abutments of existing bridges and calculating afflux in designing new bridges. It is necessary to be familiar with the rationale of these formulae to be able to apply them intelligently.

17.2. Broad Crested Weir Formula applied to Bridge Openings.

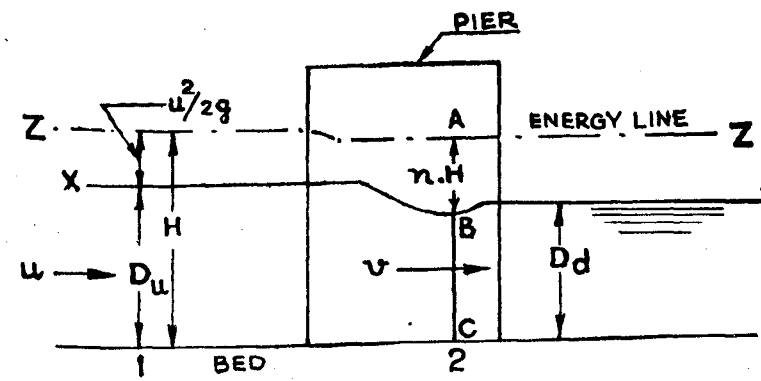


Fig. 17.2.

In Fig. 17.2, XX is the water surface profile, and ZZ the total energy line.

At Section 1, the total energy,

$$H = \frac{u^2}{2g} + D_u \quad \dots \quad (i)$$

At Section 2, let the velocity head AB be a fraction n of H, i.e.,

$$AB = \frac{v^2}{2g} = n.H. \quad \dots \quad (ii)$$

Ignoring the loss of head due to entry and friction, and equating total energies at Sections 1 and 2 :

$$H = AC = AB + BC = nH + BC$$

Hence, the depth $BC = (1-n)H$ ... **(iii)**

The area of flow at Section 2,

$$a = BC \times \text{Linear waterway}$$
$$= (1-n)\, H \times L.$$

Velocity at 2, from (ii),

$$v = (2gnH)^{\frac{1}{2}}$$

Therefore, the discharge through the bridge

$$Q = a.v.$$
$$= (1-n)\, H.L. \times (2gnH)^{\frac{1}{2}}$$

To account for losses in friction a co-efficient C_w may be introduced. Thus,

$$Q = C_w\,(1-n)\, H.L\,(2gnH)^{\frac{1}{2}}$$
$$= C_w \sqrt{2g}.L.H^{\frac{3}{2}}\,(n^{\frac{1}{2}} - n^{\frac{3}{2}}) \qquad \textbf{(iv)}$$

The depth BC adjusts itself so that the discharge passing through it is maximum. In that condition,

$$\frac{dQ}{dn} = O$$

i.e., $\frac{1}{2}.\, n^{-\frac{1}{2}} - \frac{3}{2}\, n^{\frac{1}{2}} = O$, i.e., $n = \frac{1}{3}$

Putting $n = \frac{1}{3}$ in (iv)

$$Q = 3.08\, C_w\, L.H^{\frac{3}{2}} \qquad \ldots \textbf{(17.2}_a\textbf{)}$$

Combining with (i),

$$Q = 3.08\, C_w.L\left(D_u + \frac{u^2}{64}\right)^{\frac{3}{2}} \qquad \ldots \textbf{(17.2}_b\textbf{)}$$

Since AB is $\frac{1}{3}H$, therefore BC is $\frac{2}{3}$ H, or 66·7 per cent of H.

On exit from the bridge, some of the velocity head is re-converted into potential head due to the expansion of the section and the water surface is raised, so that D_d is somewhat greater than BC, i.e., greater than 66·7 per cent of H. In fact, observations have proved that, in the limiting condition, D_d can be 80

per cent of D_u. Hence, the following rule :

"So long as the afflux $(D_u - D_d)$ is not less than $\frac{1}{4}$ D_d, the weir formula applies, i.e., Q depends on D_u and is independent of D_d."

The fact that the downstream depth D_d has no effect on the discharge Q, nor on the upstream depth D_u, when the afflux is not less than $\frac{1}{4}$ D_d, is due to the formation of the "Standing Wave".

The co-efficient C_w may be taken as under :

(1) Narrow bridge opening with or without floors ..	0·94
(2) Wide bridge opening with floors.	0·96
(3) Wide bridge opening with no bed floors ...	0·98

17.3. The Orifice Formula.

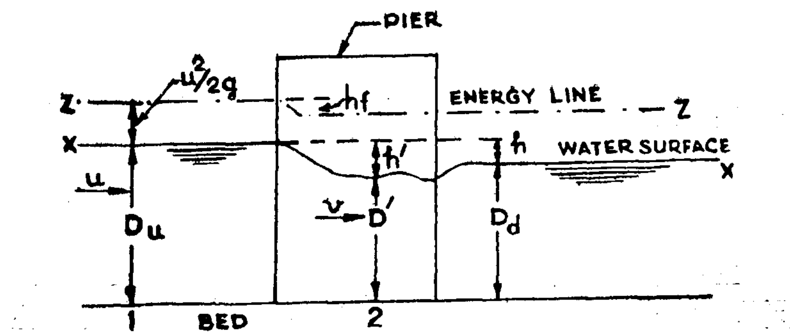


Fig. 17.3$_a$

When the downstream depth D_d is more than 80 per cent of the upstream depth D_u, the Weir Formula does not hold good, i.e., the performance of the bridge opening is no longer unaffected by D_d.

In Fig. 17·3$_a$, XX is the water surface line and ZZ the total energy line.

Apply Bernouli's Equation to points 1 and 2, ignoring the loss of head (h) due to entry and friction.

$$D_u + \frac{u^3}{2g} = D' + \frac{v^2}{2g}$$

or, $$\frac{v^2}{2g} = D_u - D' + \frac{u^2}{2g}$$

$$v = \sqrt{2g}\left(D_u - D' + \frac{u^2}{2g}\right)^{\frac{1}{2}}$$

Put $D_u - D' = h'$.

Then, $v = \sqrt{2g}\left(h' + \frac{u^2}{2g}\right)^{\frac{1}{2}}$

The discharge through the Section 2,

$Q = a.v.$

$$= L.D'.\sqrt{2g}\left(h' + \frac{u^2}{2g}\right)^{\frac{1}{2}} \qquad \text{(ii)}$$

Now the fractional difference between D' and D_a is small. Put D_a for D' in (i).

$$Q = L.D_a.\ \sqrt{2g}.\ \left[h' + \frac{u^2}{g}\right]^{\frac{1}{2}} \qquad \text{(ii)}$$

In the field it is easier to work in terms of h $(= D_u - D_a)$ instead of h'. But h is less than h', as on emergence from the bridge the water surface rises, due to recovery of some velocity energy as potential head. Suppose $\frac{e.u^2}{2ge}$ represents the velocity energy that is converted into potential head.

Then $h' = h + \frac{eu^2}{2g}$

Substituting in (ii)

$$Q = L.D_a\ \sqrt{2g}\left(h + \overline{e+1}\,\frac{u^2}{2g}\right)^{\frac{1}{2}}$$

Now introduce a coefficient C_e to account for losses of head through the bridge. We get:

$$Q = C_o.\ \sqrt{2g}.\ L.D_a\left(h + \overline{1+e}\,\frac{u^2}{2g}\right)^{\frac{1}{2}} \qquad (17.3)$$

For values of e and C_o see Figures 17.3_b and 17.3_c [10].

— COEFFICIENT "e" — FOR — THE ORIFICE FORMULA —

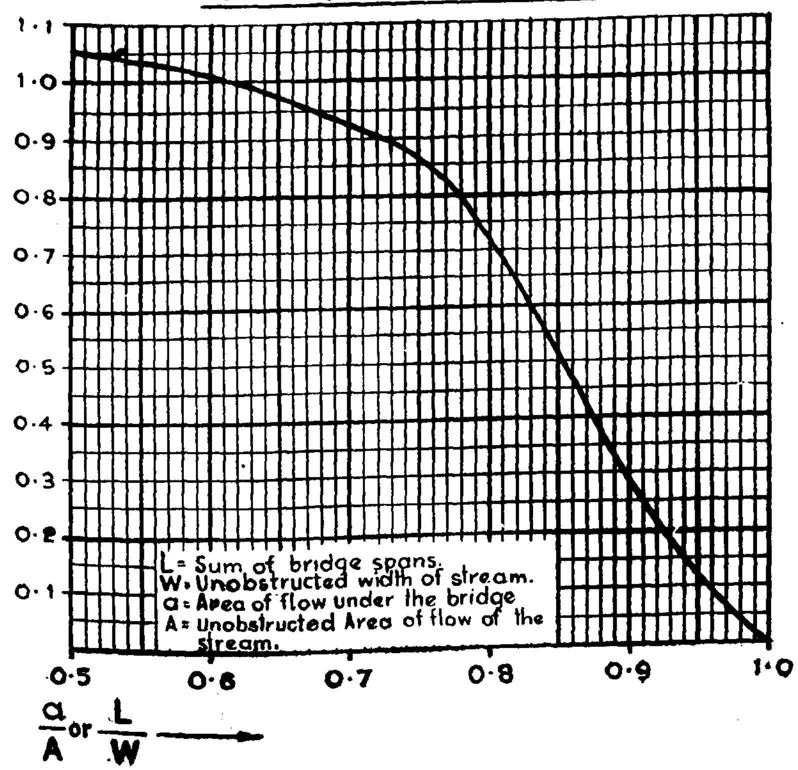


Fig. 17.3_b

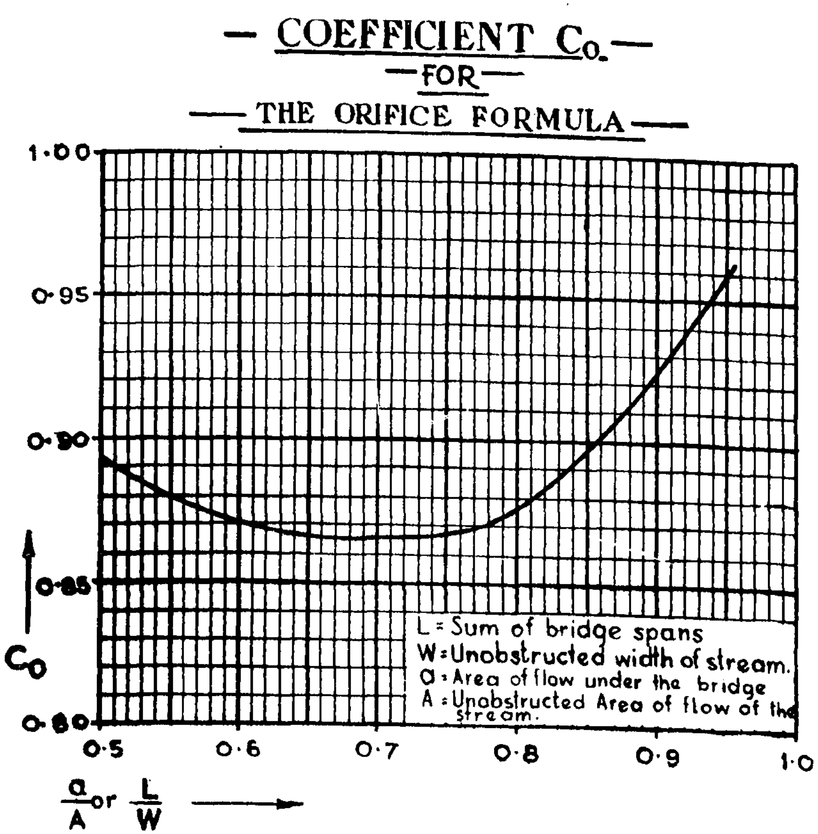


Fig. 17.3c

17.4. In Conclusion. Let us get clear on some important points :

(1) In all these formulae D_d is not affected in any way by the existence of the bridge. It depends only on the conveyance factor and slope of the tail race. D_d has therefore, got to be actually measured or calculated from Area-slope data of the channel as explained already in Article 11.

(2) The Weir Formula applies only when a standing wave is formed i.e., when the afflux ($h = D_u - D_d$) is not less than $\frac{1}{4}$ D_d.

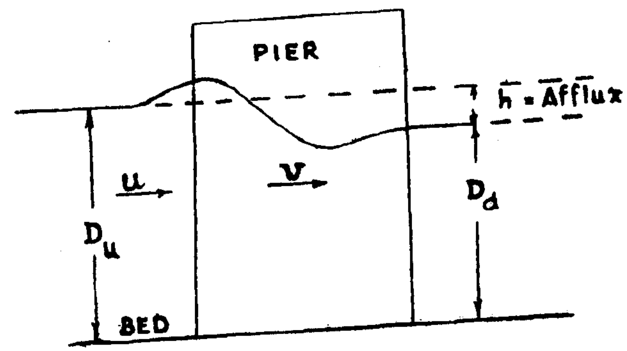
(3) The Orifice Formula with the suggested values of C_o and e should be applied when the afflux is less than $\frac{1}{4}$ D_d.

17.5. Examples have been worked out in Articles 18 and 19 to show how these formulae can be used to calculate afflux and discharge under bridges.

ARTICLE 18

AFFLUX

18.1. The afflux at a bridge is the heading up of the water surface caused by it. It is measured by the difference in levels of the water surfaces upstream and downstream of the bridge (Fig. 18.1).



SECTION

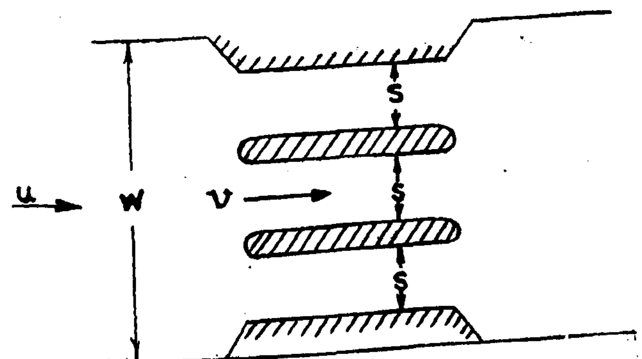


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Fig. 18.1.

18.2. When the waterway area of the openings of a bridge is less than the unobstructed natural waterway area of the stream i.e., when the bridge contracts the stream, afflux occurs. Contraction of the stream is normally not done; but under some circumstances it is taken recourse to, if it leads to ponderable economy. Also, in the case of some alluvial streams in plains the natural stream width may be much in excess of that required for regime. When spanning such a stream, it has to be contracted to, more or less, the width required for stability by providing training works.

18.3. Estimating afflux is necessary to see its effect on the 'clearance' under the bridge; on the regime of the channel upstream of the bridge; and on the design of training works.

18.4. For calculating afflux we must know (1) the discharge Q, (2) the unobstructed width of the stream W, (3) the linear waterway of the bridge, L (4) and the average depth downstream of the bridge D_d.

18.5. The downstream depth D_d is not affected by the bridge; it is controlled by the conveyance factor and slope of the channel below the bridge. Also the depth, that prevails at the bridge site before the construction of the bridge, can be assumed to continue to prevail just downstream of the bridge after its construction. Thus D_d is the depth that prevails at the bridge site before its construction. To estimate afflux we must know D_d. In actual problems, D_d is either given or can be calculated from the data supplied.

18.6. Example. A bridge, having a linear waterway of 75 ft, spans a channel 100 ft wide carrying a discharge of 2500 cusecs. Estimate the afflux when the downstream depth is 3.0 ft.

D_d = 3 ft.; W = 100 ft; L = 75 ft

Discharge through the bridge by the Orifice Formula,

$$Q = C_o \sqrt{2g}.\ L.\ D_d \sqrt{\left(h + \overline{1+e}\,\frac{u^2}{2g}\right)}$$

$$\frac{L}{W} = \frac{75}{100} = 0.75.$$

Corresponding to this, $C_o = 0.867$, $e = 0.86$

$$2500 = 0.867 \times 8 \times 75 \times 3 \times \sqrt{\left(h + 1.86\frac{u^2}{2g}\right)}$$

$$\therefore\ h + 0.029\,u^2 = 2.56 \qquad \ldots \qquad \ldots \qquad \text{(i)}$$

Also, just upstream of the bridge

$$Q = W \times (D_d + h) \times u$$

$$2500 = 100\,(3 + h) \times u$$

$$\text{or} \quad h = \frac{25}{u} - 3 \qquad \ldots \qquad \ldots \qquad \text{(ii)}$$

Substituting for h from (ii) in (i) and rearranging

$$u - 0.0052\,u^3 = 4.52. \quad \therefore\ u = 5.2 \text{ ft/sec.}$$

Substituting for u in (i)

$$h = 1.78 \text{ ft}$$

Alternatively.

Assume that h is more than $\frac{1}{4}\,D_d$

and apply the Weir Formula

$$Q = 3.08 \times C_w.L.H^{\frac{3}{2}}$$

$$2500 = 3.08 \times 0.92 \times 75\, H^{\frac{3}{2}}$$

$$\therefore\ H = 5.15 \text{ ft.}$$

$$\text{Now } H = D_u + \frac{u^2}{2g} = D_u \text{ (approx.)}$$

$$\therefore\ D_u = 5.15 \text{ ft approx.}$$

Now, $Q = W \times D_u \times u$ i.e., $2500 = 100 \times 5.15 \times u$

$$\therefore\ u = 4.85;\ \frac{u^2}{2g} = 0.37 \text{ ft}$$

$$H = D_u + \frac{u^2}{2g}$$

$$\text{i.e.,}\quad 5.15 = D_u + 0.37$$

$$D_u = 4.78 \text{ ft}$$

$$h = D_u - D_d = 4.78 - 3.0 = 1.78 \text{ ft}$$

Since h is actually more than $\frac{1}{4}\,D_d$ therefore the assumption made was correct and the application of the Weir Formula was justified.

18.7. Example. The unobstructed cross sectional area of flow of a stream is 2850 s. ft and the width of flow is 300 ft. A bridge of 4 spans of 60 ft clear is proposed across it. Calculate the afflux when the discharge is 2,300 cusecs.

$W = 300$ ft ; $L = 240$ ft ; $D_a = \frac{2850}{300} = 9.5$ ft

The depth before the construction of the bridge is the depth downstream of the bridge after its construction. Hence $D_a = 9.5$ ft

$\frac{L}{W} = \frac{240}{300} = 0.8.$ Therefore $C_o = 0.877$ and $e = 0.77.$

By the orifice formula the discharge through the bridge

$$23{,}000 = 0.877 \times 8 \times 240 \times 9.5 \times \sqrt{\left(h + 1.77\frac{u^2}{2g}\right)}$$

$$= 16000\sqrt{(h + 0.0277\,u^2)}$$

i.e., $h + 0.0277\,u^2 = 2.06$ ... ... (i)

Now, the discharge just upstream of the bridge

$$23000 = 300 \times (9.5 + h)u$$

or $h = \frac{76.67}{u} - 9.5$ .... (ii)

Putting for h from (ii) in (i) and rearranging

$$u - 0.00239.u^3 = 6.64$$

$$\therefore\ u = 7.75 \text{ ft/ sec.}$$

Putting for u in (ii)

$$h = 0.4 \text{ ft}$$

18.8. Example. A bridge 5 of spans of 120 ft each is proposed across a stream, whose unobstructed width is 830 ft, slope 1/2000 and discharge 64,000 cusecs. Calculate the afflux ($n = 0.03$).

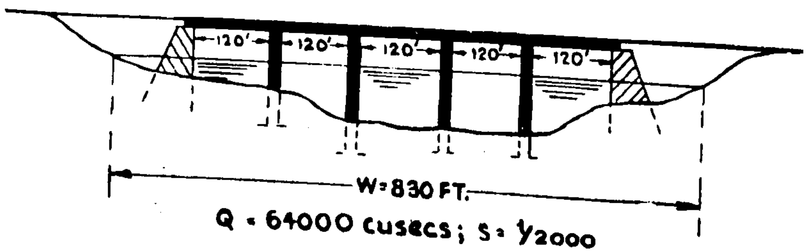


Fig. 18.8.

We have first to find D_a.

$$Q = A.V = (R.P)\,V = R.W.V.$$

$$\therefore\ R.V. = \frac{Q}{W} = \frac{64000}{830} = 77$$

Knowing $n = 0.03$; $S = 1/2000$, read velocity for various values of R from Plate 3 and select that pair whose product is 77. Thus we get

$$R = 12.7$$

$$V = 6.1$$

Take $D_a = R = 12.7$ ft

Now, $W = 830$ ft ; L 600 ft. $D_a = 12.7$ ft

$\frac{L}{W} = \frac{600}{830} = 0.72$ therefore $C_o = 0.87$; $e = 0.90$

By the orifice formula, the discharge through the bridge

$$64{,}000 = 0.87 \times 8 \times 600 \times 12.7\left(h + 1.90\frac{u^2}{2g}\right)$$

i.e., $h + 0.0292\,u^2 = 1.46$ ....(i)

The discharge just upstream of the bridge

$$64000 = 830.\,(12.7 + h).u$$

i.e., $h = \frac{77}{u} - 12.7$ ... .(ii)

Put for h from (ii) in (i) and rearrange

$u - 0.00206\,u^3 = 5.44.$ $\therefore\ u = 5.85$ ft/sec,

Put this value of u in (ii), we get :

$$h = 13.15 - 12.7 = \underline{0.45 \text{ ft}}$$

ARTICLE 19

WORKED OUT EXAMPLES ON DISCHARGE PASSED BY EXISTING BRIDGES FROM FLOOD MARKS

19.1. Calculating Discharge by the weir Formulae.

Example. The unobstructed width of a stream is 230 ft. The linear waterway of a bridge across it is 154 ft. In a flood, the average depth of flow downstream of the bridge was 11.5 ft and the afflux was 3.15 ft. Calculate the discharge.

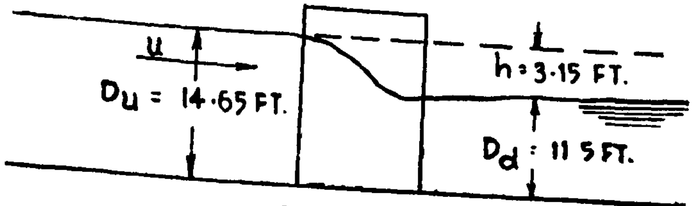


Fig. 19.1.

$$\frac{h}{D_d} = \frac{3.15}{11.15} = 0.282$$

Since h is more than 0.25 D_d, therefore the Weir Formula will apply.

W 230 = ft ; L = 154 ft ; h = 3.15 ft

Let the velocity of approach be u ft/Sec. The discharge at a section just up stream of the bridge (Fig 19.1).

$$Q = u \times 14.65 \times 230 = 3373\ u \ldots\ldots\ldots\ldots(i)$$

The discharge through the bridge by the Weir Formula

$$Q = 3.08 \times 0.98 \times 154\left(14.65 + \frac{u^2}{64}\right)^{\frac{3}{2}}$$

$$= 466\left(14.65 + \frac{u^2}{64}\right)^{\frac{3}{2}}$$

Equating values of Q from (i) and (ii)

$$3370\ u = 466\left(14.65 + \frac{u}{64}\right)$$

Rearranging $u^{\frac{3}{2}} - 0.00417\ u^2 = 3.92$

or u = 8.7 ft/sec.

Putting the value of u in (i) or (ii) we get Q.

Q = 3370 × 8.7

= 29,300 cusecs.

Try the Orifice Formula.

$$\frac{L}{W} = \frac{154}{230} = 0.67 \qquad \therefore C_o = 0.865 ;\ e = 0.95$$

Discharge through the bridge by the Orifice Formula

$$Q = 0.85 \times 8 \times 154 \times 11.5 \times \sqrt{\left(3.15 + 1.95 \times \frac{u^2}{64}\right)}$$

$$= 12{,}250 \sqrt{(3.15 + \cdot0304\ u^2)} \ldots\ldots\ldots\ldots(i)$$

Discharge just upstream of the bridge

$$Q = 230 \times 14.65 \times u$$

$$= 3370\ u \ldots\ldots\ldots\ldots(ii)$$

Equating values of Q in (i) and (ii)

$$12250 \sqrt{(3.15 + \cdot0304\ u^2)} = 3370\ u$$

Simplifying and squaring

$$u^2 = \frac{41.6}{0.6}. \qquad \therefore u = 8.35 \text{ ft/sec.}$$

Substituting for u in (i) or (ii) we get Q

Q = 3370 × 8.35 = 28,200 cusecs.

This result is about the same as given by the first method. In fact, the orifice formula, with the recommended values of C_o and e gives nearly correct results even where the conditions are appropriate for the Weir Formula. But the converse is not true.

19.2. Calculating Discharge by the Orifice Formula.

Example. The unobstructed width of a stream is 425 ft and the linear waterway of a bridge across it 350 ft. During a flood the average depth of flow downstream of the bridge was 8·5 ft and the afflux 0.35 ft. Calculate discharge.

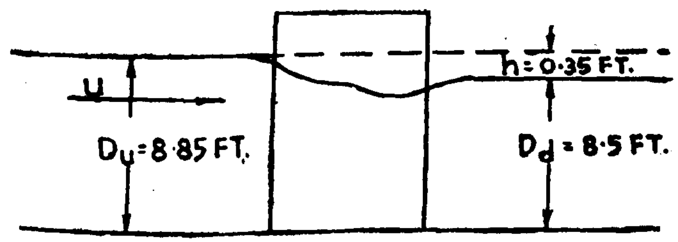


Fig. 19.2.

Given: W = 425 ft L = 350 ft h = 0.35 ft Let the velocity of approach be u. The discharge at a section just upstream of the bridge

$$Q = u \times 8.85 \times 425 = 3761\,u \ldots\ldots\ldots\ldots(\text{i})$$

$$\text{Contraction} = \frac{a}{A} = \frac{L}{W} = \frac{350}{425} = 0.825$$

Corresponding to this, $C_o = 0.88$ and $e = 0.63$. The discharge under the bridge, by the Orifice Formula,

$$Q = 0.88 \times 8 \times 8.5 \times 350 \times \sqrt{\left(0.35 + \frac{1.63u^2}{64}\right)}$$

$$= 20900 \sqrt{(0.35 + 0.0255\ u^2)} \ldots\ldots\ldots\ldots(\text{ii})$$

Equating values of Q in (i) and (ii)

$$3761\,u = 20900 \times \sqrt{(0.35 + 0.0255)u^2}$$

Squaring and rearranging

$$u^2 = \frac{10.8}{0.21} = 51.4. \qquad \therefore\ u = 7.18 \text{ ft/sec.}$$

Substitute for u in (i) or (ii) to get Q.

$$Q = 3761 \times 7.18 = 27{,}000 \text{ cusecs.}$$

19.3. The Border-line Cases. An example will now follow to illustrate what results are obtained by applying the Weir Formula and the Orifice Formula to cases which are on the border line *i. e.*, where the afflux is just $\frac{1}{4}\ D_d$.

19·4. Example. A stream, whose unobstructed width is 750 ft is spanned by a bridge whose linear waterway is 650 ft. During a flood the average downstream depth was 8.5 ft and the afflux was 2.1 ft. Calculate the discharge

$$\frac{h}{D_d} = \frac{2.1}{8.5} = 0.247$$

Since h is nearly $\frac{1}{4}\ D_d$ therefore both the Weir Formula and Orifice Formula should apply.

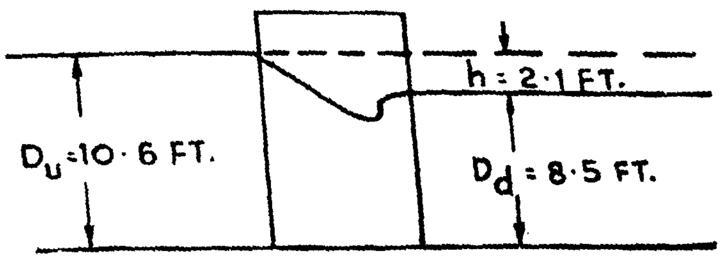


Fig. 19.4.

By the Weir Formula.

If the velocity of approach is u, the discharge just upstream of the bridge

$$Q = 750 \times 10.6 \times u = 7950\,u \ldots\ldots\ldots\ldots(\text{i})$$

The discharge through the bridge

$$Q = 3.08 \times 0.98 \times 650\left(10.6 + \frac{u^2}{64}\right)^{\frac{3}{2}} \ldots\ldots\ldots\ldots(\text{ii})$$

Equating values of Q from (i) and (ii)

$$7950\,u = 1960\left(10.6 + \frac{u^2}{64}\right)^{3}$$

Rearranging

$$u^{\frac{2}{3}} - 0.00615\,u^2 = 4.18. \qquad \therefore\ u = 10.75 \text{ ft/sec}$$

Put for u in (i)

$$Q = 7950 \times 10.75 = \mathbf{85{,}500\ cusecs.}$$

By the Orifice Formula.

$$\frac{a}{A} = \frac{L}{W} = \frac{650}{750} = 0.865 \qquad e = 0.44.$$

$$\therefore \quad C_o = 0.9.$$

If u is the velocity of approach, the discharge just upstream of the bridge

$$Q = 750 \times 10.6 \times u = 7950\,u \ldots\ldots\ldots\ldots(\text{i})$$

The discharge under the bridge by the Orifice Formula,

$$Q = 0.9 \times 8 \times 650 \times 8.5\left(2.1 + \frac{1.44\,u^2}{64}\right)^{\frac{1}{2}} \ldots\ldots\ldots\ldots(\text{ii})$$

$$= 39{,}800\ (2.1 + 0.0225\,u^2)^{\frac{1}{2}}$$

Equating values of Q from (i) and (ii)

$$7950\,u = 39{,}800(2.1 + 0.0225\,u^2)^{\frac{1}{2}}$$

$$u = 5.0(2.1 + 0.0225\,u^2)^{\frac{1}{2}}$$

Squaring and rearranging—

$$u^2 = \frac{52.5}{0.4375} = 120\,;\ u = 11.0 \text{ ft/sec.}$$

Substituting for u in (i) or (ii) we get Q.

$$Q = 7950 \times 11 = \mathbf{87{,}450\ cusecs.}$$

ARTICLE 20

OVERTOPPING OF THE BANKS

20.1. In plains where the ground slopes are gentle and the natural velocities of flow in streams are low, the flood water may spill over one or both the banks of the stream at places.

20.2. Height of Approach Roads. Consider the case where the bulk of the discharge is carried by the main channel and a small fraction of it flows over the banks somewhere upstream of the bridge. If the overflow strikes high ground at a short distance from the banks, it can be forced back into the stream and made to pass through the bridge. This can be done by building the approach roads of the bridge solid and high so that they intercept the overflow. In this arrangement, the linear waterway of the bridge must be ample to handle the whole discharge without detrimental afflux. Also the top level of the approach roads must be high enough not to be overtopped. If the velocity of the stream is V, the water surface level, where it strikes the road embankment will be $\frac{V^2}{64}$ ft. higher than the H.F.L. in the stream at the point where the overflow starts. This arrangement is therefore, normally feasible where the stream velocity is not immoderately high.

20.3. Subsidiary or Relief Culverts. Sometimes, however, the overflow spreads far and away from the banks. This is not infrequently the case in alluvial plains, where the ground level falls continuously away from the banks of the stream. In such cases it is impossible to force the overflow back into the main stream. The correct thing to do is to pass the overflow through relief culverts at suitable points in the road embankment. These culverts have to be carefully designed. They should not be too small to cause detrimental ponding up of the overflow, resulting in damage to the road or some property. Nor, should they be so big as to attract the main current.

20.4. Permanent Dips and Breaching Sections in Approach Roads. It is sometimes feasible as well as economical to provide permanent dips (or alternatively breaching sections) in the bridge approaches to take excessive overflows in emergencies. The dips or breaching sections have to be sited and designed so

that the velocity of flow through them does not become erosive, cutting deep channels and ultimately leading to the shifting of the main current.

20.5. Retrogression of Levels. Suppose water overflows a low bank somewhere upstream of the bridge and, after passing through a relief culvert, rejoins the main stream somewhere lower down. When the flood in the main channel subsides, the ponded up water at the inlet of the subsidiary culvert gets a free over fall. Under such conditions deep erosion can take place. A deep channel is formed, beginning at the outfall in the main stream and retrogressing towards the culvert. This endangers the culvert. To provide against this, protection has to be designed downstream of the culvert so as to dissipate the energy of the falling water on the same lines as is done on irrigation "falls". That is, a suitable cistern and baffle wall should be added for dissipating the energy and the issuing current should be stilled through a properly designed expanding flume.

ARTICLE 21

FEASIBILITY OF PIPE AND BOX CULVERTS FLOWING FULL

21.1. Many a plain is one vast flat without any deep and defined drainage channels in it. When the rain falls, the surface water moves in some direction in a wide sheet of nominal depth. So long as this movement of water is unobstructed no damage may occur to property or crops. But when a road embankment is thrown across the country intercepting the natural flow, water ponds up on one side of it. Relief has then to be afforded from possible damage from this ponding up by taking the water across the road through causeways or culverts.

21.2. In such contourless regions the road runs across wide but shallow dips and therefore the most straightforward way of handling the surface flow is to provide suitable dips (i.e., causeways) in the longitudinal profile of the road and let water pass over them.

21.3. There may, however, be cases where the above solution is not the best. Some of its limitations may be cited. Too many causeways or dips detract from the usefulness of the road. Also, the flow of water over numerous sections of the road, makes its proper maintenance problematic and expensive. Again, consider the case of a wet cultivated or waterlogged country (and flat plains are not infrequently swampy and water-logged) where the embankment has necessarily got to be taken high above the ground. Frequent dipping down from high road levels to the ground produces a very undesirable road profile. And, even cement concrete slabs, in dips across a waterlogged country, do not rest evenly on the mud underneath them. Thus, it will appear that constructing culverts in such circumstances should be a better arrangement than providing dips or small causeways.

21.4. After we have decided that a culvert has to be constructed on a road lying across some such country, we proceed to calculate the discharge by using one of the run-off formulae, having due regard to the nature of the terrain and the intensity of rainfall as already explained in Article 4.

But the natural velocity of flow cannot be estimated because (i) if there is no defined cross section of the channel from which we may take the area of cross section and wetted perimeter; and (ii) there is no measurable slope in the drainage line. Even where we could calculate or directly observe the velocity, it may be so small that we could not aim at passing water through the culvert at that velocity, because the area of waterway required for the culvert $\left(A = \frac{Q}{V}\right)$ will be prohibitively large. In such cases the design has to be based on an increased velocity of flow through the culvert and to create that velocity the design must provide for heading up at the inlet end of the culvert. Economy in design being the primary consideration, the correct practice, indeed, is to design a pipe or a box culvert on the assumption that water at the inlet end may head up to a predetermined safe level above the top of the inlet opening. This surface level of the headed up water at the upstream end has to be so fixed that the road bank should not be overtopped, nor any property in the flood plain damaged.

Next, the level of the downstream water surface should be noted down. This will depend on the size and the slope of the leading out channel and is, normally, the surface level of the natural unobstructed flow at the site, that prevails before the road embankment is constructed.

After this we can calculate the required area of cross section of the barrel of the culvert by applying the principles of Hydraulics discussed in Article 22.

21.5. The procedure set out above is rational and considerable research has been carried out on the flow of water through pipe and box culverts, flowing full.

21.6. In the past, use was extensively made of empirical formulae which gave the ventway area required for a culvert to

drain a given catchment area. Dun's Drainage Table is one of that class and is purely empirical. This Table is still widely used, as it saves the trouble of hydraulic calculations. But it is unfortunate that recourse is often taken rather indiscriminately to such short cuts, even where other more accurate and rational, procedure is possible and warranted by the expense involved. Dun's Table, or others in that class, should NOT be used until suitable correction factors have been carefully evolved from extensive observations (in each particular region with its own singularities of terrain and climate) of the adequacy or otherwise of the existing culverts vis-a-vis their catchment areas.

21.7. Considerations of economy require that small culverts, in contrast with relatively larger structures across defined channels, need not be designed normally to function with adequate clearance for passing floating matter. The depth of a culvert should be small and it does not matter if the opening stops appreciably below the formation level of the road. Indeed, it is correct to leave it in that position and let it function even with its inlet submerged. This makes it possible to design low abutments supporting an arch or a slab, or alternatively, to use round pipes or square box barrels.

21.8. Nor should high headwalls be provided for retaining deep over-fills; instead, the length of the culverts should be increased suitably so that the road embankment, with its natural side slopes, is accommodated without high retainiug headwalls.

21.9. Where masonry abutments supporting arches or slabs are designed for culverts functioning under "head," bed pavement must be provided. And, in all cases, including pipe and box culverts, adequate provision must be made at the exit against erosion by designing curtain walls. Where the exit is a free overfall a suitable cistern and baffle wall must be added for the dissipation of energy and stilling of the ensuing current.

ARTICLE 22

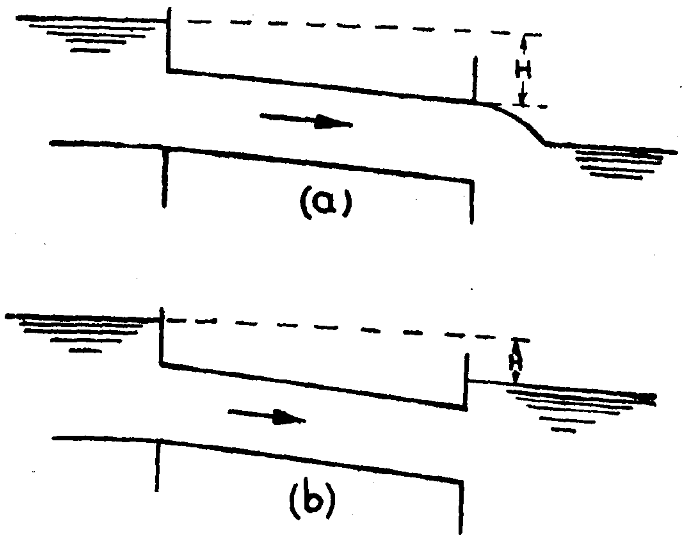
HYDRAULICS OF THE PIPE AND BOX CULVERTS FLOWING FULL

22.1. The Permissible Heading up at the Inlet. It has been explained already that where a defined channel does not exist and the natural velocity of flow is very low, it is economical to design a culvert as consisting of a pipe or a number of pipes of circular or rectangular section functioning with the inlet submerged. As the flood water starts heading up at the inlet, the velocity through the barrel goes on increasing. This continues till the discharge passing through the culvert equals the discharge coming towards the culvert. When this state of equilibrium is reached the upstream water level does not rise any higher.

For a given design discharge, the extent of upstream heading up depends on the ventway of the culvert. The latter has to be so chosen that the heading up should not go higher than a predetermined safe level, the criterion for safety being that the road embankment should not be overstopped, nor any property damaged by submergence. The fixing of this level is the first step in the design.

22.2. Surface Level of the Tail Race. It is essential that the H. F. L. in the outfall channel near the exit of the culvert should be known. This may be taken as the H. F. L. prevailing at the proposed site of the culvert before the construction of the road embankment with some allowance for the concentration of flow caused by the construction of the culvert.

22.3. The Operating Head When the Culverts Flow Full. In this connection the cases that have to be considered are illustrated in Figure 22.3. In each case the inlet is submerged



THE OPERATING HEAD "H" WHEN THE CULVERT FLOWS FULL.
Fig. 22.3,

and the culvert is flowing full. In case (a) the tail race water surface is below the crown of the exit and in case (b) it is above that. The operating head in each case is marked "H". Thus we see that: "When the culvert flows full, the operating head, H, is the height of the up stream water level measured from the surface level in the tail race or from the crown of the exit of the culvert whichever is higher."

22.4. The Velocity Generated by "H". The operating head "H" is utilized in (i) supplying the energy required to generate the velocity of flow through the culvert, (ii) forcing water through the inlet of the culvert, and (iii) overcoming the frictional resistance offered by the inside wetted surface of the culvert.

If the velocity through the pipe is v, the head expended in generating it is $\frac{v^2}{2g}$. As regards the head expended at the entry, it is customary to express it as a fraction K_e of the velocity head $\frac{v^2}{2g}$ Similarly, the head required for overcoming the friction of the pipe is expressed as a fraction Kf of $\frac{v^2}{2g}$. From this it follows, that

$$H = [1 + K_e + K\ \ ]\frac{v^2}{64} \qquad \ldots \qquad (22{\cdot}4)$$

From this equation we can calculate the velocity v, which a given head H will generate in a pipe flowing full, if we know K_e and Kf.

22.5. Values of K_e and Kf. K_e principally depends on the shape of the inlet. The following values are commonly used:

$$\left.\begin{aligned} K_e &= 0.08 \text{ for bevelled or bell-mouthed entry} \\ &= 0{\cdot}505 \text{ for sharp edged entry} \end{aligned}\right\} \qquad (22{\cdot}5\ a)$$

As regards Kf it is a function of the length L of the culvert, its hydraulic mean radius R, and the coefficient of rugosity n of its surface.

The following relationship exists between K_f and n.

$$Kf = \frac{22n^2}{R^{\frac{1}{3}}}\cdot\frac{L}{R} \qquad (22{\cdot}5b)$$

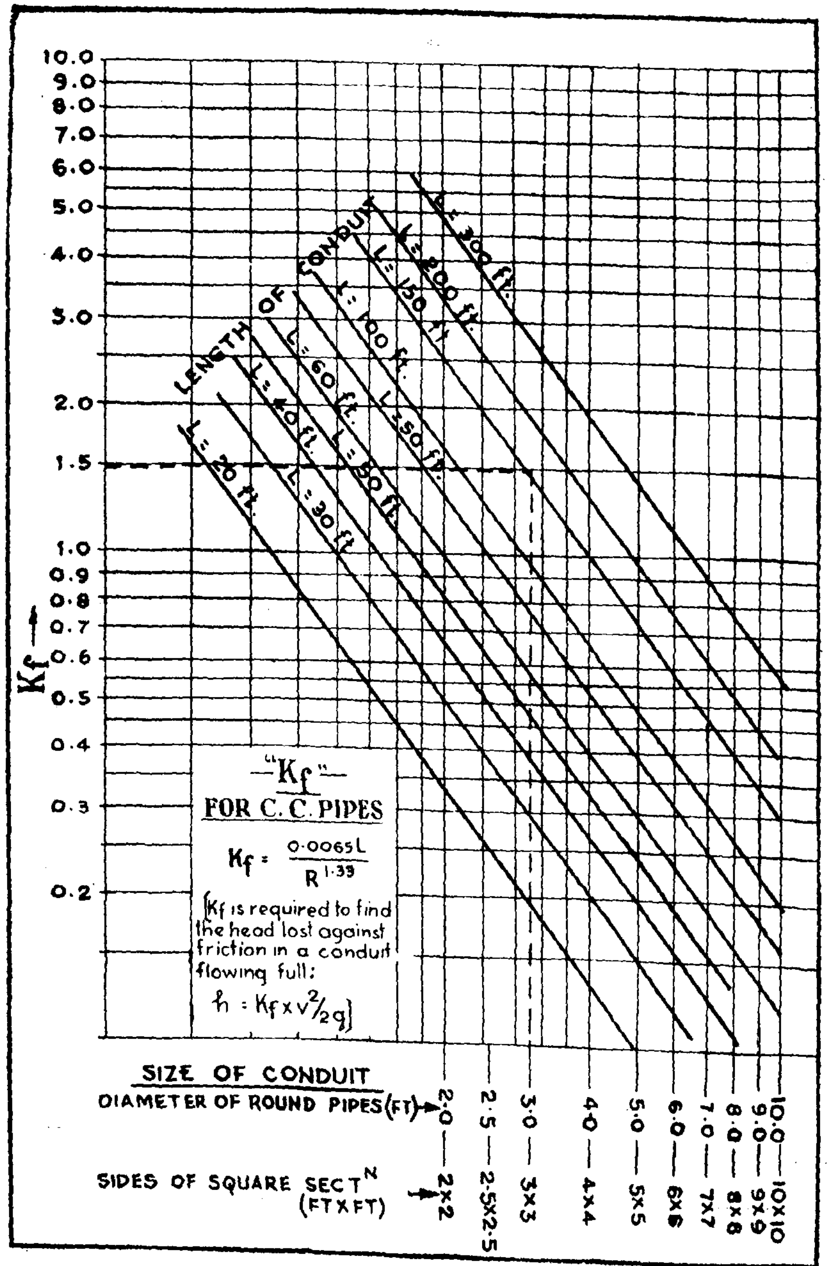


Fig. 22.5,

For cement concrete circular pipes or cement plastered masonry culverts of rectangular section, the coefficient of rugosity $n = 0{\cdot}015$, therefore the above equation reduces to :

$$K_f = \frac{0{\cdot}0065\,L}{R^{1.33}} \quad \ldots \quad (22{\cdot}5_c)$$

The graphs in Fig. 22·5 are based on Equation $22{\cdot}5_c$. For a culvert of known sectional area and length, K_f can be directly read from these graphs.

22.6. Values of K_e and K_f Modified through Research. Considerable research has recently been carried out on the head lost in flow through pipes. The results have unmistakably demonstrated the following:

The entry loss coefficient, K_e, depends not only on the shape of the entry but also on the size of entry and the roughness of its wetted surface. In general K_e increases with an increase in the size of the inlet.

Also K_f, the friction loss coefficient, is not independent of K_e. Attempts to make the entry efficient repercuss adversely on the frictional resistance to flow offered by the wetted surface of the barrel. In other words, if the entry conditions improve (i.e. if K_e decreases), the friction of the barrel increases (i.e., K_f increases). This phenomenon can be explained by thinking of the velocity distribution inside the pipe. When the entry is square and sharp edged, high velocity lines are concentrated nearer the axis of the barrel; while the bell mouthed entry gives uniform distribution of velocity over the whole section of the barrel. From this it follows that, the average velocity being the same in both cases, the velocity near the wetted surface of the pipe will be lower for square entry than for bell mouthed entry. Hence, the frictional resistance inside the culvert is smaller when the entry is square than when it is bell mouthed. Stream lining the entry is, therefore, not an unmixed advantage.

Consequently, it has been suggested that the vales of K_e and K_f should be as given in Table 22.6.

TABLE 22.6.

Values of K_e and K_f [9]

Entry & Friction Co-efficients	Circular Pipes.		Rectangular Culverts.	
	Square entry	Bevelled entry	Square entry	Bevelled entry
$K_e =$	$0.61\ R^{0.5}$	0.1	$0.4R^{0.3}$	0.05
$K_f =$	$0.005L/R^{1.2}$	$0.0005L/R^{1.2}$	$0.0045\ L/R^{1.25}$	$0.0045L/R^{1.25}$

22.7. Design Calculations.

We have said that

$$H = \left[1 + K_e + K_f\right]\frac{v^2}{64}$$

$$\text{i.e., } v = 8\left(\frac{H}{1 + K_e + K_f}\right)^{\frac{1}{2}}$$

Hence,

$$Q = A \times 8\left(\frac{H}{1 + K_e + K_f}\right)^{\frac{1}{2}} \qquad \textbf{(22.7)}$$

Suppose we know the operating head H and the length of the barrel L; and assume that the diameter of a round pipe or the side of a square box culvert is D.

From D calculate the cross sectional area A and the hydraulic mean radius R of the culvert.

Now, from R and L compute K_e and K_f, using appropriate functions from Table 22.6. Then, calculate Q from Equation (22.7). If this equals the design discharge, the assumed size of the culvert is correct. If not, assume a fresh value of D and repeat.

22.8. Design Chart (PLATE 6.)

Equation (22.7) may be written as

$$Q = \lambda \sqrt{2gH} \qquad (22.8_a)$$

$$A$$

$$\text{where } \lambda = (1 + K_e + K_f)^{\frac{1}{2}} \qquad (22.8_b)$$

It is obvious that all components of λ in Equation (22.8_b) are functions of the cross section, length, roughness, and the shape of the inlet of the pipe. Therefore λ represents the conveying capacity of the pipe and may be called the 'Conveyance Factor'. The discharge, then, depends on the conveyance factor of the pipe and the operating head. In **Plate** 6 curves have been constructed from Equation (22.8_a) from which Q can be directly read for any known values of λ and H.

Also, in the same **Plate,** Tables are included from which λ can be taken for any known values of (1) length, (2) diameter in case of circular pipes or sides in case of rectangular pipes, and (3) conditions of entry, viz., sharp-edged or round. The material assumed is cement concrete and values of K_e and Kf used in the computation are based on functions in Table (22.6).

The use of PLATE 6 renders the design procedure very simple and quick. Examples will now follow to illustrate.

22.9. Example. Data: full with bevelled entry.

(1) Circular Cement concrete pipe flowing

(2) Operating head = 3.0 ft

(3) Length of the pipe = 70 ft

(4) Diameter = 3.5 ft

Find the discharge.

See, in **Plate 6,** the Table for Circular Pipes with Rounded Entry.

For L = 75 ft and D = 3.5 ft, the conveyance factor

$\lambda = 7.8$.

Now refer to the curves in the same **Plate.** F[illegible]λ = 7.8 and H = 3.0 ft

Q = 108

22.10. Example. Design a culvert consisting of Cement Concrete circular pipes with bevelled entry and flowing full, given: (see Fig. 22.10)

Discharge	=	370 cusecs
R. L. of ground	=	100.00
H. F. L. of tail race	=	102.500
Permissible heading up at inlet R. L.	=	105.50
Length of Culvert	=	60 ft.

Since we shall try pipes of diameters exceeding 2.5 ft, the culvert will function as sketched:

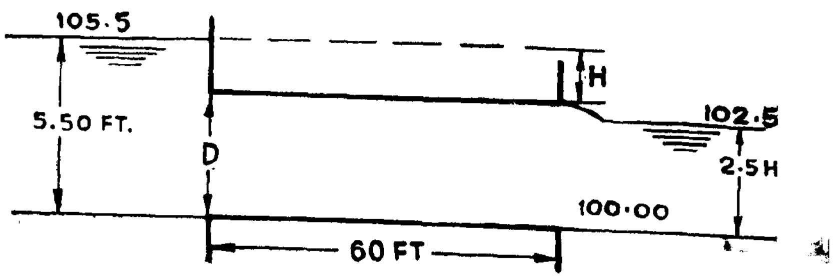


Fig. 22.10.

Assumed value of D	=	(1) 3.0 ft;	(2) 3.5 ft;	(3) 4.0 ft
Corresponding $H = 5.5 - D$	=	(1) 2.5 ft	(2) 2.0 ft;	(3) 1.5 ft
Discharge per pipe from PLATE 6, Q,	=	(1) 70 cus.;	(2) 92 cus.;	(3) 105 cus.;
Number of pipes required $370/Q$	=	(1) 5.3; say 5	(2) 4.0; say 4	(3) 3.5 say 4

Hence 4 pipes of 3.5 ft diameter will suit.

Acknowledgments

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HIGHE

Appendix

	March	:tober	November	December	Highest intensity
	0.75 1st 1952 (18-19)	0.36 ɛh 1948 17-18)	0.14 24th 1951 (19-20)	0.01 31st 1952 (6-7)	3.12 7th Sept. 1948 (8-9)
	1.16 21st 1950 20-21	0.59 2h 1949 19-20	0.19 21st 1948 23-24	0.28 24th 1950 16-17	2.86 11th Aug. 1951 15-16
ı	0.08 25th 1951 15-16	0.00 1	0.17 23rd 1951 13-14	0.03 31st 1952 18-19	1.96 17th July 1949 14-15
:	0.20 25th 1951 0-1	0.03 h 1952 20-21	0.00	0.00	1.74 17th July 1950 15-16
	0.39 15th 1952 22-23	0.00 :	0.33 25th 1951 8-9	0.29 29th 1951 20-21	1.55 10th Aug. 1952 13-14
	0.20 14th 1950 20-21	0.09 t 1950 22-23	0.00	0.02 24th 1950 20-21 & 22-23	2.00 5th July 1950 12-13
)	0.40 21st 1946 10-11	3.12 h 1947 2-3	1.56 20th 1946 23-24	1.72 1st 1952 16-17	3.12 4th Oct 1947 2-3
1	1.32 9th 1911 17-18	2.00 st 1952 14-15	2.44 8th 1916 15-16	1.04 8th 1942 18-19	3.00 26th Sep. 1903 23-24
:	1.32 25th 1951 23-24	2.58 h 1953 18-19	1.14 16th 1948 23-24	0.72 6th 1946 19-20	2.58 9th Oct. 1953 18-19
2	0.01 22nd 1951 17-18	2.03 th 1950 21-22	0.83 25th 1951 8-9	0.00	3.00 23rd July 1951 11-12
3	0.34 15th 1952 18-19	1.46 th 1953 8-9	0.70 23rd 1952 23-24	0.00	1.99 7th June 1953 4-5
	...	...	...	...	1.58 18th July 1953 23-24
	...	...	...	...	...
0	0.59 28th 1951 16-17	0.85 th 1949 6-7	0.49 5th 1948 15-16	0.07 30th 1950 10-11	1.99 24th Aug. 1952 15-16

HIGHEST INTENSITY OF RAINFALL IN INCHES

Station		January	February	March	April	May	June	July	August	September	October	November	December	Highest intensity
1. New Delhi (Safdarjung)	Amt	1.13	0.32	0.75	0.20	0.53	0.48	1.96	1.48	3.12	0.36	0.14	0.01	3.12
	Date	15th 1953	4th 1949	1st 1952	2nd 1951	30th 1950	24th 1953	21st 1949	16th 1952	7th 1948	30th 1948	24th 1951	31st 1952	7th Sept. 1948
	Time	(18-19)	(1-2)	(18-19)	(8-9)	(15-16)	(4-5)	(7-8)	(15-16)	(8-9)	(17-18)	(19-20)	(6-7)	(8-9)
2. Allahabad (Bamrauli)	Amt	0.18	0.49	1.16	0.34	0.24	2.37	1.14	2.86	1.46	0.59	0.19	0.28	2.86
	Date	25th 1950	5th 1953	21st 1950	2nd 1951	16th 1952	30th 1951	19th 1948	11th 1951	10th 1949	29th 1949	21st 1948	24th 1950	11th Aug. 1951
	Time	20-21 & 21-22	19-20	20-21	16-17	0-1	11-12	1-2	15-16	2-3	19-20	23-24	16-17	15-16
3. Jodhpur	Amt	0.52	0.16	0.08	0.08	0.70	1.10	1.96	1.60	1.20	0.00	0.17	0.03	1.96
	Date	18th 1948	20th 1948	25th 1951	18th 1953	30th 1951	27th 1951	17th 1949	13th 1951	4th 1953		23rd 1951	31st 1952	17th July 1949
	Time	16-17	23-24	15-16	20-21	3-4	17-18	14-15	6-7	16-17		13-14	18-19	14-15
4. Jaipur	Amt	0.22	0.23	0.20	0.12	0.48	1.46	1.74	1.60	0.80	0.03	0.00	0.00	1.74
	Date	11th 1953	20th 1952	25th 1951	1st 1952	21st 1950	15th 1951	17th 1950	21st 1952	8th 1950	4th 1952			17th July 1950
	Time	11-12	7-8	0-1	16-17	16-17	1-2	15-16	19-20	20-21	20-21			15-16
5. Amritsar	Amt	0.32	0.26	0.39	0.14	0.18	1.01	0.64	1.55	0.09	0.00	0.33	0.29	1.55
	Date	15th 1953	4th 1952	15th 1952	20th 1953	11th 1952	21st 1952	27th 1952	10th 1952	13th 1951		25th 1951	29th 1951	10th Aug. 1952
	Time	15-16	6-7	22-23	2-3	7-8	22-23	14-15	13-14	17-18		8-9	20-21	13-14
6. Aligarh	Amt	0.32	0.00	0.20	0.11	0.22	0.96	2.00	0.56	1.08	0.09	0.00	0.02	2.00
	Date	24th 1950		14th 1950	1st 1950	30th 1950	23rd 1950	5th 1950	30th 1950	1st 1950	1st 1950		24th 1950	5th July 1950
	Time	16-17		20-21	23-24	17-18	0-1	12-13	10-11	14-15	22-23		20-21 & 22-23	12-13
7. Madras	Amt	1.10	0.26	0.40	1.80	2.07	1.12	0.97	2.45	1.73	3.12	1.56	1.72	3.12
	Date	27th 1947	23rd 1950	21st 1946	1st 1945	19th 1952	1st 1949	9th 1946	25th 1950	23rd 1950	4th 1947	20th 1946	1st 1952	4th Oct 1947
	Time	23-24	10-11	10-11	12-13	3-4	20-21	1-2	2-3	3-4	2-3	23-24	16-17	2-3
8. Bangalore	Amt	2.02	2.34	1.32	2.40	2.76	2.40	2.35	2.79	3.00	2.00	2.44	1.04	3.00
	Date	16th 1908	21st 1901	9th 1911	9th 1943	30th 1900	1st 1952	25th 1916	5th 1948	26th 1903	21st 1952	8th 1916	8th 1942	26th Sep. 1903
	Time	0-1	20-21	17-18	18-19	19-20	22-23	1-2	21-22	23-24	14-15	15-16	18-19	23-24
9. Kodaikanal	Amt	1.40	0.67	1.32	1.76	1.58	0.98	1.02	0.92	0.82	2.58	1.14	0.72	2.58
	Date	16th 1948	21st 1952	25th 1951	4th 1952	22nd 1951	5th 1951	7th 1949	23rd 1953 & 28th 1950.	12th 1948	9th 1953	16th 1948	6th 1946	9th Oct. 1953
	Time	19-20	11-12	23-24	17-18	17-18	17-18	12-13	15-16 ; 16-17	16-17	18-19	23-24	19-20	18-19
10. Visakhapatnam	Amt	0.40	0.16	0.01	1.57	1.20	0.67	3.00	2.20	2.11	2.03	0.83	0.00	3.00
	Date	18th 1953	23rd 1952	22nd 1951	30th 1951	27th 1952	18th 1952	23rd 1951	20th 1951	22nd 1950	28th 1950	25th 1951		23rd July 1951
	Time	2-3	14-15	17-18	20-21	19-20	8-9	11-12	14-15	13-14	21-22	8-9		11-12
11. Trivandrum	Amt	0.86	0.66	0.34	1.09	1.25	1.99	0.80	0.35	0.62	1.46	0.70	0.00	1.99
	Date	24th 1953	21st 1953	15th 1952	14th 1952	10th 1952	7th 1953	6th 1953	13th 1952	4th 1953	25th 1953	23rd 1952		7th June 1953
	Time	22-23	15-16	18-19	13-14	17-18	4-5	5-6	2-3	17-18	8-9	23-24		4-5
12. Mangalore	Amt	...	...	...	0.14	0.23	1.08	1.58	0.67	0.58	...	...	...	1.58
	Date				25th 1953	26th 1953	30th 1953	18th 1953	1st 1953	4th 1953				18th July 1953
	Time				5-6	5-6	19-20	23-24	22-23	3-4				23-24
13. Tiruchirapally	Amt	...	...	....	...	...	0.29	0.33	0.60	...	...	...	...	...
	Date						26th 1953	25th 1953	27th 1953					
	Time						17-18	23-24	20-21					
14. Nagpur	Amt	0.55	0.37	0.59	0.35	0.42	1.20	1.86	1.99	1.90	0.85	0.49	0.07	1.99
	Date	4th 1948	21st 1950	28th 1951	26th 1948	27th 1949	27th 1947	21st 1951	24th 1952	5th 1951	10th 1949	5th 1948	30th 1950	24th Aug. 1952
	Time	00-01	4-5	16-17	18-19	18-19	20-21	22-23	15-16	18-19	6-7	15-16	10-11	15-16

HIGHEST INTENSITY OF RAINFALL IN INCHES—(*Contd.*)

Station		January	February	March	April	May	June	July	August	September	October	November	December	Highest intensity
15. Jagdalpur	Amt Date Time	0.44 23rd 1943 1-2	0.00	0.00	1.13 15th 1953 19-20	0.00	0.55 5th 1953 18-19	0.78 22nd 1953 21-22	1.15 12th 1953 13-14	1.45 23rd 1953 13-14	0.83 10th 1953 16-17	0.16 11th 1953 3-4	1.07 9th 1952 17-18	1.45 23rd Sept. 1953 13-14
16. Bhopal (Bairagarh)	Amt Date Time	...	...	...	0.05 8th 1952 7-8 & 8-9	0.00	2.40 18th 1953 15-16	0.62 16th 1952 15-16	0.75 5th 1953 4-5	0.71 14th 1952 18-19	0.03 14th 1952 3-4	0.00	0.28 20th 1952 20-21	2.40 18th June 1953 15-16
17. Poona	Amt Date Time	0·40 22nd 1948 8-9	0.11 24th 1938 19-20	0.18 26th 1938 & 27th 1947 15-16 ; 17-18	1.36 1953 ...	2.10 30th 1933 17-18	1.71 1953 ...	0.67 4th 1932 12-13	1.32 15th 1947 2-3	1.60 21st 1951 15-16	1·54 18th 1932 21-22	0.84 3rd 1931 4-5	0.40 1st 1932 0-1	2.10 30th May 1933 17-18
18. Calcutta (Alipore)	Amt Date Time	0.35 30th 1944 3-4	0.42 23rd 1946 7-8	0.96 13th 1948 20-21	1.73 25th 1952 19-20	1.90 21st 1947 8-9	2.22 23rd 1944 15-16	1.98 17th 1951 16-17	2.50 12th 1947 1-2	2.32 27th 1945 3-4	2.08 15th 1882 ...	0.98 1st 1950 19-20	0.14 23rd 1947 6-7	2.50 12th Aug., 1947 1-2
19. Calcutta (Dum Dum)	Amt Date Time	0.15 — 1948	1.19 1948	0.77 1948	1.60 1949	1.40 1953	2.48 1953	2.68 1953	1.77 1951	1.81 1949	1.36 1951	0.53 1950	0.00	2.68 July 1953
20. Asansol	Amt Date Time	0.54 — 1953 ...	0.18 10th 1952 12-13	0.77 2nd 1952 20-21	0.76 22nd 1952 16-17	0.80 27th 1951 21-22	1.30 22nd 1952 14-15	1.42 5th 1953 3-4	1.32 10th 1950 16-17	0.83 10th 1951 16-17	0.62 3rd 1951 15-16	0.29 11th 1953 6-7	0.00	1.42 5th July 1953 3-4
21. Chirrapunji	Amt Date Time	0.22 23rd 1953 22-23	1.00 10th 1950 4-5	2.04 30th 1951 2-3	3.60 7th 1947 23-24	3.64 11th 1948 0-1	4.28 17th 1949 6-7	5.00 11th 1952 0-1	2.96 23rd 1951 11-12	4.20 14th 1951 0-1	1.74 4th 1951 11-12	0.66 18th 1950 23-34	0.14 20th 1951 13-14	5.00 11th July 1952 0-1
22. Port Blair	Amt Date Time	0.37 1953	1.02 1953	0.52 1952	0.40 1952 & 1953	2.00 1952	1.40 1953	1.18 1951	2.33 1953	1.54 1951	2.30 1951	0.80 1951	0.72 1951	2.33 Aug. 1953
23. Saugor Island	Amt Date Time	0.50 1953	1.36 6th 1948 9-10	0.62 10th 1952 18-19	1.92 1949	1.64 21st 1940 22-23	2.16 10th 1950 10-11	2.54 1951	3.00 18th 1952 9-10	2.72 1951	1.29 11th 1952 5-6	1.07 18th 1950 8-9	0.63 1947	3.00 18th Aug. 1952 9-10
24. Jamshedpur	Amt Date Time	0.60 27th 1949 20-21	0-41 4th 1948 15.16	0.76 24th 1951 18-19	0.81 11th 1949 21-22	2.14 20th 1949 15-16	3.38 10th 1949 0-1	1.94 17th 1953 2-3	2.43 29th 1953 21-22	1.68 14th 1950 23-24	0.94 1st 1947 14-15	0.70 25th 1948 23-24	0.11 14th 1947 11-12	3.38 10th June 1949 0-1
25. Gaya	Amt Date Time	0.46 27th 1949 18-19	0.25 5th 1949 19-20	0.58 24th 1948 19-20	0.46 19th 1953 17-18	1.02 12th 1946 22-23	1.64 22nd 1953 15-16	1-76 6th 1947 16-17	2.40 31st 1946 0-1	1.85 13th 1948 4-5	1.00 10th 1948 18-19	0.50 27th 1948 4-5	0.16 28th 1952 2-3	2.40 31st Aug. 1946 0.1
26. Agartala	Amt Date Time	...	0.87 1953	0.74 1953	—	0.83 1953	...	...	1.40 1953	1.46 1953	1.55 1953	...	...	1.55 Oct. 1953
27. Mohanbari (Dibrugarh)	Amt Date Time	—	...	...	—	...	1.40 1953	...	1.69 1953	1.24 1953	0.65 1953	...	...	...
28. Bokaro	Amt Date Time	0.11 11th 1952 9-10	0.04 10th & 22nd 1952 7-8 ; 5-6	0.60 16th 1952 11-12	1.28 19th 1951 17-18	1.08 26th 1951 18-19	0.98 28th 1952 23-24	1.89 11th 1951 18-19	1.14 17th 1951 2-3	1.89 10th 1952 13-14	1.44 29th 1951 17-18	0.02 25th 1951 21-22	0.00	1.89 11th July 1951 & 10th Sept. 1952 18-19 ; 13-14

HIGHEST INTENSITY OF RAINFALL IN INCHES—(*Contd.*)

Station		January	February	March	April	May	June	July	August	September	October	November	December	Highest Intensity
29. Hazaribagh	Amt	0.17	0.10	0.49	0.80	1.00	0.48	1.37	1.66	0.94	0.19	0.00	0.02	1.66
	Date	11th 1952	10th 1952	2nd 1952	15th 1952	28th 1952	26th 1952	16th 1952	19th 1952	10th 1952	9th 1952		28th 1952	19th August 1952
	Time	4–5	7–8	17–18	14–15	14–15	15–16	18–19	22–23	23–24	15–16		7–8	22–23
30. Dhanbad	Amt	0.08	0.17	0.14	0.24	0.61	1.24	1.93	1.24	1.35	1.20	0.00	0.00	1.93
	Date	11th 1952	10th 1952	16th 1952	14th 1952	12th 1952	15th 1952	24th 1952	6th 1952	7th 1952	13th 1952			24th July 1952
	Time	10–11	8–9	12–13	16–17	3–4	10–11	19–20	6–7	1–2	11–12			19–20
31. Bombay (Colaba)	Amt	1.84	0.49	0.11	0.11	2.17	4.22	2.68	2.14	5.06	1.70	1.25	0.46	5.06
	Date	1926	1937	1951	1951 & 53	1943	11th 1847	1949	1935	1949	11th 1948	1927 & 22nd 1948	1933	Sept. 1949
	Time										7–8	– ; 3–4		
32. Bombay (Santacruz)	Amt	0.00	0.00	0.00	0.00	0.00	3.28	3.60	0.83	1.08	1.73	0.00	0.00	3.60
	Date						1953	1952	1953	1953	1952			July 1952
	Time													
33. Veraval	Amt	0.00	0.00	0.00	0.00	0.00	0.85	1.00	0.98	0.94	0.00	0.00	0.00	1.00
	Date						24th 1952	18th 1952	1953	1953				18th July 1952
	Time						12–13	9–10						9–10
34. Vengurla	Amt	0.00	Trace	0.00	0.39	0.50	1.80	1.66	0.60	0.53	0.97	0.00	0.03	1.80
	Date		1952		18th 1952	25th 1952	4th 1952	1953	1953	1953	7th 1952		9th 1952	4th June 1952
	Time				18–19	10–11	6–7				22–23		20–21	6–7
35. Ahemedabad	Amt	0.14	0.00	0.00	0.00	0.02	1.20	1.39	1.04	2.08	0.00	0.00	0.00	2.08
	Date	1953				1953	1952	1953	17th 1951	1953				Sept. 1953
	Time								16–17					
36. Baroda	Amt	0.17	0.00	0.00	0.00	0.00	0.95	1.11	1.49	1.34	0.00	0.00	0.00	1.49
	Date	1953					1953	11th 1950	1953	16th 1950				August 1953
	Time							23–24		23–24				
37. Chikaltana	Amt	Trace	0.10	0.00	0.16	0.34	1.42	1.32	0.22	1.46	0.25	0.39	0.05	1.46
	Date	1953	9th 1952		1953	22nd 1952	1953	1953	6th 1952	20th 1952	1st 1952	1951	13th 1952	20th Sept. 1952.
	Time		16–17			22–23			16–17	22–23	15–16		20–21	16–17
38. Hyderadad (Begumpet)	Amt	0.14	0.82	0.40	0.55	0.79	1.69	1.72	1.17	0.92	0.92	0.00	0.06	1.72
	Date	1953	20th 1950	26th 1951	19th 1951	29th 1950	9th 1952	1953	1953	10th 1950	14th 1951		11th 1952	July 1953
	Time		14–15	17–18	12–13	19–20	0–1			20–21	14–15		14–15	
39. Mahabaleshwar	Amt	0.20	0.02	0.76	0.18	0.88	2.00	1.32	1.00	1.59	1.78	1.14	0.04	2.00
	Date	22nd 1948	21st 1948	6th 1948	14th 1951	1949	23rd 1951	20th 1950	1946	1950	1st 1951	20th 1951	26th 1950	23rd June 1951
	Time	10–11	14–15	17–18	15–16		23–24	6–7			15–16	21–22	11–12	23–24

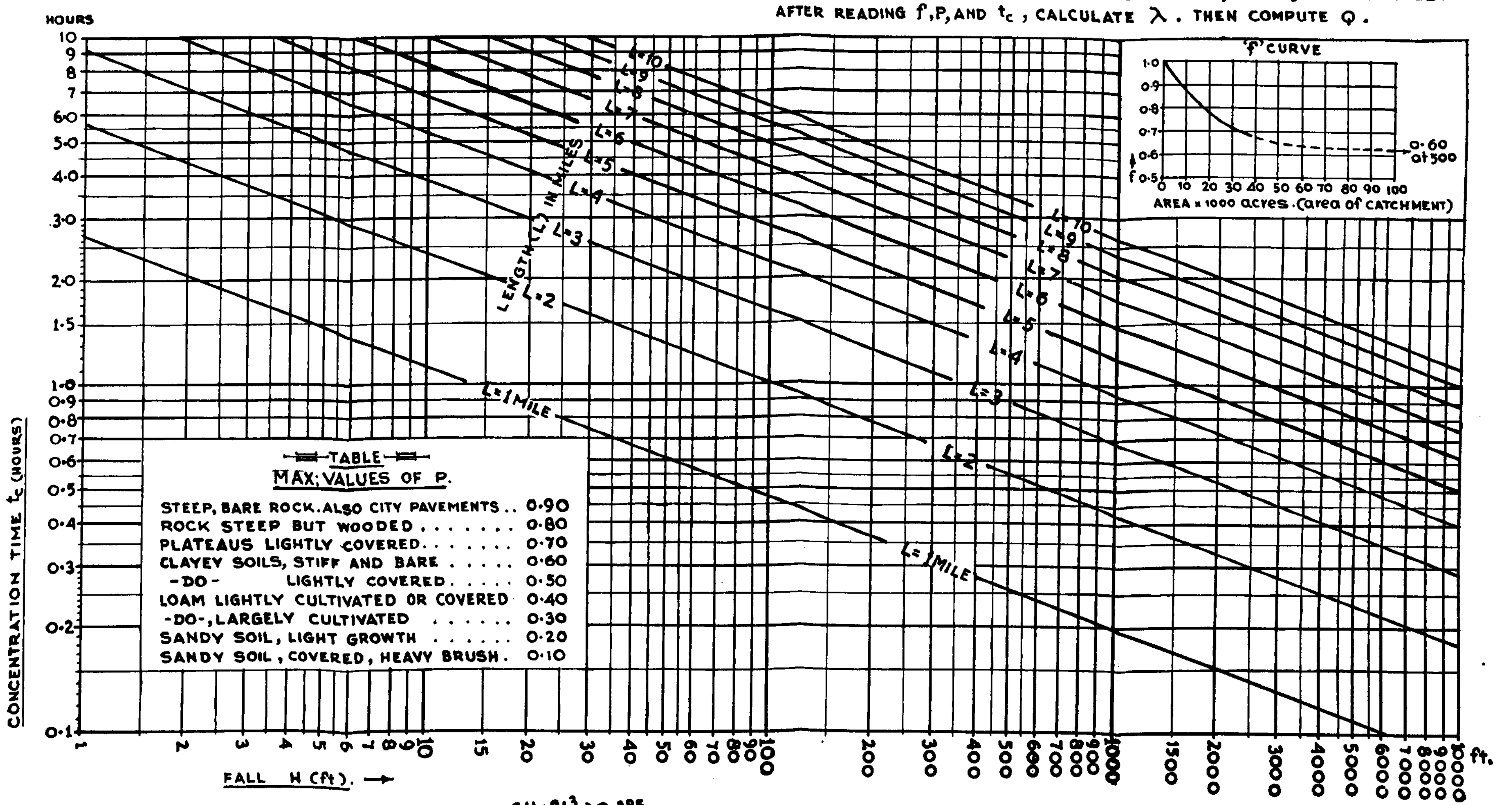
RUN OFF DATA

$$Q = A \cdot I_0^{\frac{2}{3}} \cdot \lambda$$

Q = RUN OFF IN CUSECS; A = AREA OF CATCHMENT IN ACRES; I_0 = THE MAX: ONE HOUR STORM PRECIPITATION IN INCHES FOR THE REGION.

$$\lambda = \frac{2 f \cdot P}{t_c + 1}$$

FOR f SEE INSET CURVE; FOR P SEE INSET TABLE; SEE t_c FROM CURVES. AFTER READING f, P, AND t_c, CALCULATE λ. THEN COMPUTE Q.



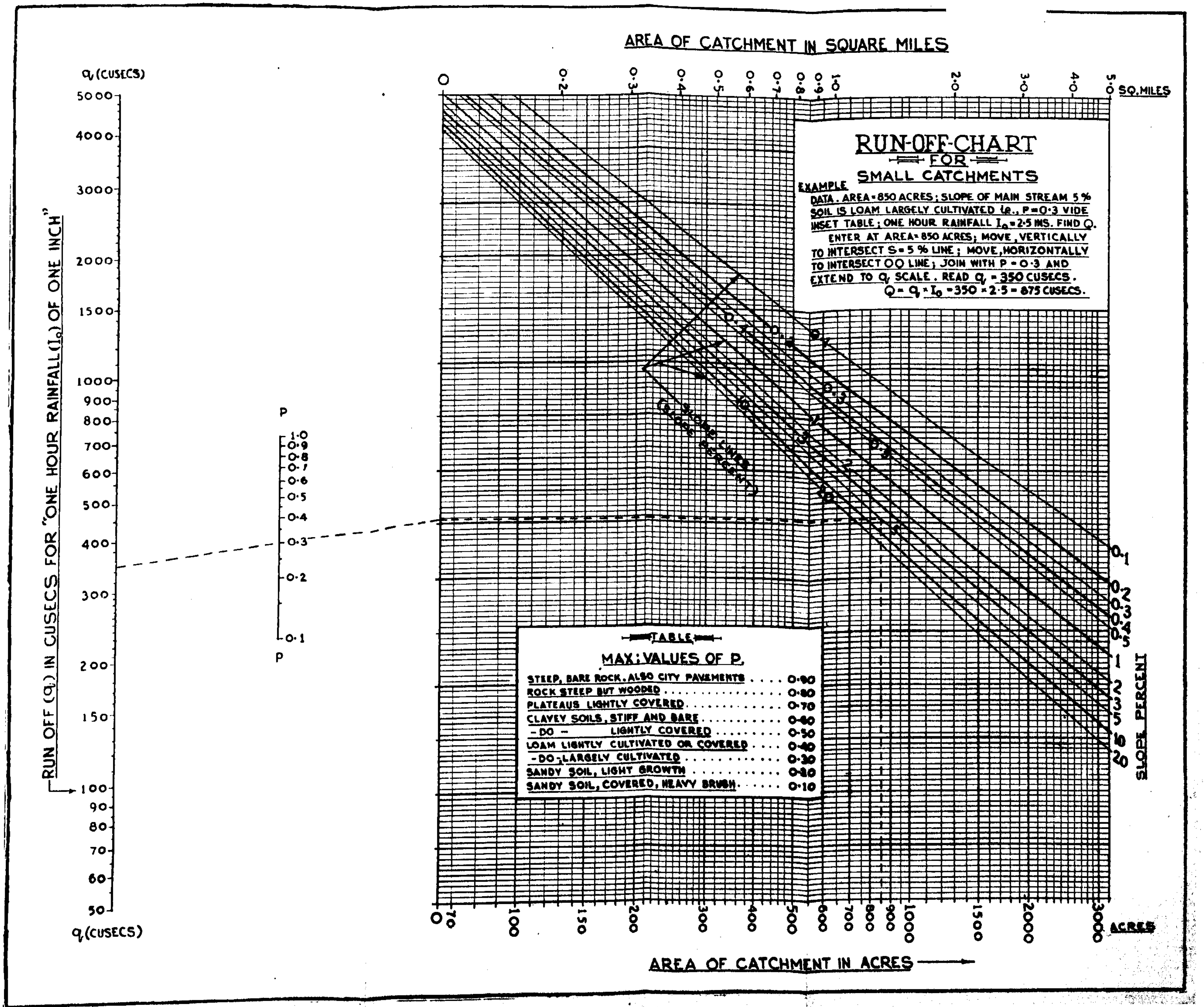
TABLE

MAX; VALUES OF P.

STEEP, BARE ROCK. ALSO CITY PAVEMENTS ..	0·90
ROCK STEEP BUT WOODED	0·80
PLATEAUS LIGHTLY COVERED.......	0·70
CLAYEY SOILS, STIFF AND BARE	0·60
-DO- LIGHTLY COVERED.....	0·50
LOAM LIGHTLY CULTIVATED OR COVERED	0·40
-DO-, LARGELY CULTIVATED	0·30
SANDY SOIL, LIGHT GROWTH	0·20
SANDY SOIL, COVERED, HEAVY BRUSH.	0·10

TIME OF CONCENTRATION IN HOURS, $t_c = \left(\frac{11 \cdot 9 L^3}{H}\right)^{0 \cdot 385}$; WHERE L = LENGTH OF CHANNEL IN MILES FROM THE CRITICAL POINT TO THE CULVERT; H = FALL IN FEET FROM THE CRITICAL POINT TO THE CULVERT.

CURVES FOR TIME OF CONCENTRATION


AREA OF CATCHMENT IN SQUARE MILES
0 0·2 0·3 0·4 0·5 0·6 0·7 0·8 0·9 1·0 2·0 3·0 4·0 5·0 SQ. MILES
RUN-OFF-CHART
FOR
SMALL CATCHMENTS
EXAMPLE
DATA. AREA = 850 ACRES; SLOPE OF MAIN STREAM 5 %
SOIL IS LOAM LARGELY CULTIVATED i.e., P = 0·3 VIDE
INSET TABLE; ONE HOUR RAINFALL I_o = 2·5 INS. FIND Q.
ENTER AT AREA = 850 ACRES; MOVE VERTICALLY
TO INTERSECT S = 5 % LINE; MOVE HORIZONTALLY
TO INTERSECT OO LINE; JOIN WITH P = 0·3 AND
EXTEND TO q_1 SCALE. READ q_1 = 350 CUSECS.
$Q = q_1 \times I_o = 350 \times 2·5 = 875$ CUSECS.
q_1 (CUSECS)
5000 4000 3000 2000 1500 1000 900 800 700 600 500 400 300 200 150 100 90 80 70 60 50
RUN OFF (q_1) IN CUSECS FOR "ONE HOUR RAINFALL (I_o) OF ONE INCH"
q_1 (CUSECS)
P
1·0 0·9 0·8 0·7 0·6 0·5 0·4 0·3 0·2 0·1
P
SLOPE LINES (SLOPE PERCENT)
0·1 0·2 0·3 0·4 0·5 1 2 3 5 10 20
SLOPE PERCENT
TABLE
MAX: VALUES OF P.
STEEP, BARE ROCK, ALSO CITY PAVEMENTS .... 0·90
ROCK STEEP BUT WOODED .... 0·80
PLATEAUS LIGHTLY COVERED .... 0·70
CLAYEY SOILS, STIFF AND BARE .... 0·60
- DO - LIGHTLY COVERED .... 0·50
LOAM LIGHTLY CULTIVATED OR COVERED .... 0·40
- DO - LARGELY CULTIVATED .... 0·30
SANDY SOIL, LIGHT GROWTH .... 0·20
SANDY SOIL, COVERED, HEAVY BRUSH .... 0·10
0 70 100 150 200 300 400 500 600 700 800 900 1000 1500 2000 3000 ACRES
AREA OF CATCHMENT IN ACRES

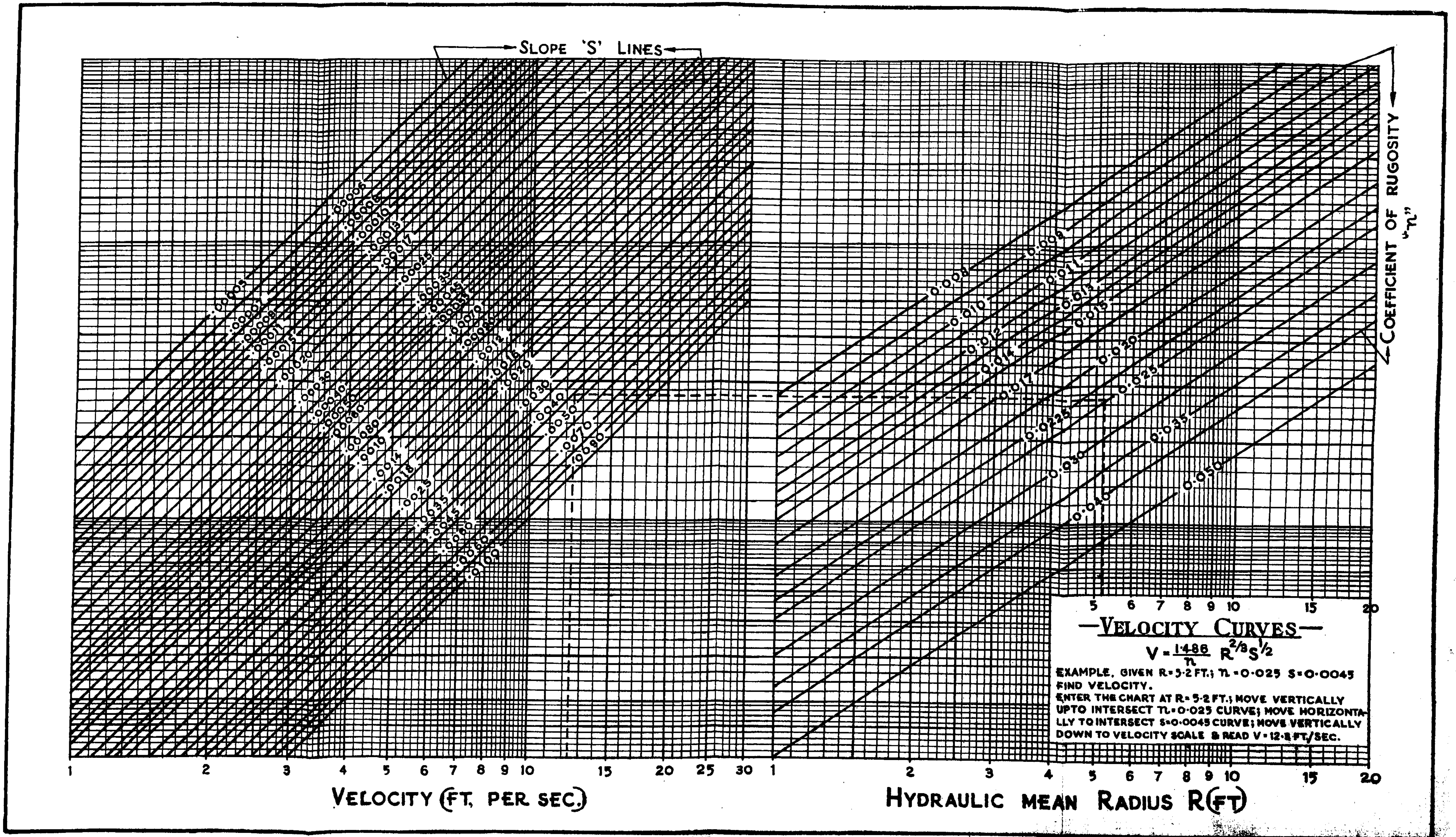
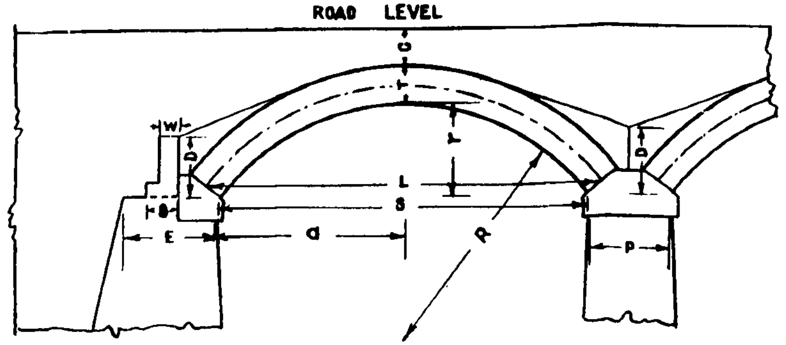

SLOPE 'S' LINES
COEFFICIENT OF RUGOSITY "n"
VELOCITY (FT. PER SEC.)
HYDRAULIC MEAN RADIUS R(FT)
—VELOCITY CURVES—
V = 1·486/n R^2/3 S^1/2
EXAMPLE. GIVEN R = 5·2 FT.; n = 0·025 S = 0·0045
FIND VELOCITY.
ENTER THE CHART AT R = 5·2 FT.; MOVE VERTICALLY
UPTO INTERSECT n = 0·025 CURVE; MOVE HORIZONTALLY TO INTERSECT S = 0·0045 CURVE; MOVE VERTICALLY
DOWN TO VELOCITY SCALE & READ V = 12·8 FT./SEC.

Plate 4

MASONRY ARCHES FOR HIGH LEVEL BRIDGES.

S = clear span.
L = effective span = S + 0·8T.
a = half span = S/2.
r = rise
R = radius of curvature of intrados
$= \frac{a^2 + r^2}{2r}$
T = thickness of arch at crown.
C = cover over crown of arch up to road level.



D = depth of haunch filling at pier & abutment = $\frac{r+T}{2}$

Widths of wall to retain haunch filling:-

Span	B = Base width Stone	B = Base width Brick	W = Top width Stone	W = Top width Brick
Up to 10′	1′-0″	1′-1½″	1′-0″	1′-1½″
20′	1′-6″	1′-6″	1′-0″	1′-1½″
30′	1′-9″	1′-10½″	1′-0″	1′-1½″
40′	2′-0″	2′-3″	1′-0″	1′-1½″

E = width of abutment and abutment pier at springing.
P = width of pier at springing.

EFFECTIVE SPAN IN FEET (L)	4		8		10		15		20		25		30		35		40	
ARCH THICKNESS AT CROWN & COVER OVER CROWN FOR :	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.	THICKNESS FT. IN.	COVER FT. IN.
(a) 1st. CLASS DRESSED STONE OR CEMENT CONCRETE (1:2½:5) BLOCK MASONRY IN CEMENT MORTAR (1:3)	1—0	1—6	1—0	2—0	1—0	2—0	1—3	2—0	1—6	2—0	1—8	2—3	1—9	2—6	1—10	2—6	2—0	2—6
(b) 1st. CLASS DRESSED STONE OR CEMENT CONCRETE (1:2½:5) BLOCK MASONRY IN LIME MORTAR (1:2)	1—0	1—6	1—0	2—0	1—3	2—0	1—6	2—0	1—9	2—0	1—10	2—3	2—0	2—6	2—3	2—6	2—6	2—6
(c) 1st. CLASS BRICK MASONRY IN CEMENT MORTAR (1:3)	1—1½	1—6	1—1½	2—0	1—1½	2—0	1—6	2—0	1—10½	2—0	1—10½	2—3	2—3	2—6	2—3	2—6	2—7½	2—6
(d) 1st. CLASS BRICK MASONRY IN LIME MORTAR (1:2)	1—1½	1—6	1—1½	2—0	1—6	2—0	1—10½	2—0	1—10½	2—0	2—3	2—0	2—7½	2—0	3—0	2—0	3—4½	2—0
(e) LIME CONCRETE (STONE METAL & HYDRAULIC LIME MORTAR) AT CROWN	1—3	1—6	1—6	2—0	1—8	2—0	1—9	2—0	2—0	2—0	2—3	2—0	2—9	2—0				
AT SPRINGING	1—3		1—6		1—8		2—0		2—3		2—6		3—0					
WIDTH AT SPRINGING:- ABUTMENTS & ABUTMENT PIERS (E)	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.	FT.	IN.
(a) 1st. CLASS STONE MASONRY IN CEMENT MORTAR (1:3)	2	6	3	3	3	6	4	3	5	0	5	9	6	6	7	3	8	0
(b) 1st. CLASS BRICK MASONRY IN CEMENT MORTAR (1:3)	2	7½	3	4½	3	9	4	6	5	3	6	0	6	9	7	6	8	3
PIERS (P)																		
(a) 1st. CLASS STONE MASONRY IN CEMENT MORTAR (1:3)	1	8	2	3	2	6	2	9	3	3	3	9	4	3	5	0	5	9
(b) 1st. CLASS STONE MASONRY IN LIME MORTAR (1:2)	1	10	2	6	2	9	3	0	3	3	3	9	4	3	5	0	5	9
(c) 1st. CLASS BRICK MASONRY IN CEMENT MORTAR (1:3)	1	10½	2	3	2	7½	3	0	3	4½	4	1½	4	6	5	3	6	0
(d) 1st. CLASS BRICK MASONRY IN LIME MORTAR (1:2)	1	10½	2	7½	3	0	3	4½	3	9	4	1½	4	6	5	3	6	0
(e) 1st. CLASS STONE MASONRY IN LIME (1:2) FOR LIME-CONCRETE ARCHES ONLY	2	0	2	6	2	9	3	3	3	9	4	0	4	10				
(f) 1st. CLASS BRICK MASONRY IN LIME (1:2) FOR LIME-CONCRETE ARCHES ONLY	1	10½	2	7½	3	0	3	4½	3	9	4	1½	4	10½				

Note :—1. The specification for the masonry of abutments, abutment piers and piers is assumed to be the same as that used for the mansonry of the arch ring itself, except that

(i) the top of abutments and abutment piers for a depth of 2 ft shall be in cement in all cases, and

(ii) for lime concrete arches, the abutments, abutment piers and piers may be either in stone or brick masonry in lime mortar.

2. Thickness of abutments, abutment piers & piers at different depths below spring for arches of various spans are given in separate tables and graphs.

DESIGN LIVE LOAD:- I. R. C. Standard Class 'A' (Revised 1947) Wheel Load Train

SHAPE OF ARCH :- Segmental.

RISE/SPAN RATIO (R/S :-) 1/4

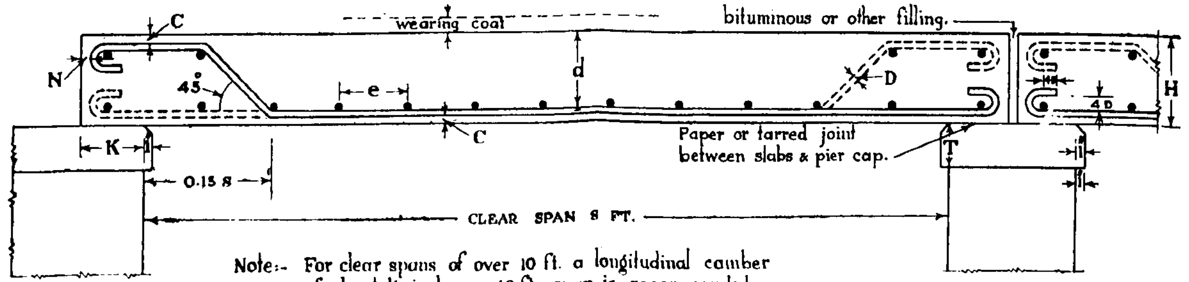
DESIGN STRESSES	SAFE STRESSES IN TONS/□′	
MATERIALS OF CONSTRUCTION:	Compressive.	Tensile.
First class dressed stone or cement concrete block masonry in cement mortar 1 : 3	15	1.3 (20 LBS./□″)
First class dressed stone or cement concrete block masonry in lime mortar 1 : 2	8–9	0.65 (10 LBS./□″)
First class brick masonry in cement mortar 1 : 3	8–9	1.3 (20 LBS./□″)
First class brick masonry in lime mortar 1 : 2	5–6	0.65 (10 LBS./□″)
Lime concrete masonry (with stone metal & hydraulic lime mortar)	4–5	0.65 (10 LBS./□″)

Plate 5

TABLES FOR FREELY SUPPORTED R. C. SLABS FOR ROAD BRIDGES & CULVERTS

CLEAR SPANS 3 FT. TO 25 FT.

C = Clear cover minimum 1".
N = End cover or clearance $1\frac{1}{2}$" or 2D whichever is more.
D = Diameter of main reinforcement bar (from tables).
e = Spacing of distribution reinforcement (from tables).
L = Effective Span S + d.



Note:- For clear spans of over 10 ft. a longitudinal camber of about ½ inch per 10 ft. span is recommended.

K = Length of bearing
= 0.5+0.04S feet or 14" whichever is less.
T = Thickness of bearing slab or pier cap.
= 0.25 + 0.02S feet.
H = Overall depth (from tables)
= d + c + D/2
d = Effective depth (from tables) measured to the centre of the main reinforcement.

TABLE A - FOR WEARING COAT. UP TO 3 INS. THICK

CLEAR SPAN	MAXIMUM BENDING MOMENT PER FOOT WIDTH	MAXIMUM SHEAR PER FOOT WIDTH	OVERALL DEPTH OF SLAB EXCLUDING WEARING COAT	EFFECTIVE DEPTH OF SLAB EXCLUDING WEARING COAT	MAIN REINFORCEMENT: DIAM. OF BARS	MAIN REINFORCEMENT: SPACING CENTRES	MAIN REINFORCEMENT: AREA PER FOOT WIDTH AT MIDSPAN	MAIN REINFORCEMENT: NO. OF MAIN BARS CRANKED UP AT EACH END	DISTRIBUTION REINFORCEMENT: DIAM.	DISTRIBUTION REINFORCEMENT: SPACING CENTRES IN THE CENTRAL PORTION OF SPAN	DISTRIBUTION REINFORCEMENT: AREA PER FOOT WIDTH
FT.	INS. LBS.	LBS.	INS.	INS.	INS.	INS.	SQ. INS.	NOS.	INS.	INS.	SQ. IN.
S	M	V	H	d	D		As			e	
3	34,000	3,970	7.00	5.75	3/8	3 ½	0.408	nil	3/8	7	0.189
4	46,000	4,440	7.00	5.75	½	4 ¼	0.554	nil	½	9¾	0.241
5	56,000	4,520	8.00	6.75	½	4	0.586	1 in every 4	½	9 ½	0.250
6	93,000	5,670	9.25	8.00	½	3	0.785	1 " " 4	½	8 ¼	0.285
7	113,000	6,050	10.00	8.69	5/8	4 ¼	0.868	1 " " 4	½	7¾	0.304
8	138,000	6,250	11.00	9.69	5/8	4	0.920	1 " " 4	½	7 ½	0.314
9	159,000	6,590	11.75	10.37	¾	5 ¼	1.010	1 " " 4	½	7 ½	0.314
10	190,000	6,660	12.75	11.37	¾	4¾	1.120	1 " " 4	5/8	11	0.335
12	250,000	7,210	14.25	12.87	¾	4 ¼	1.248	1 " " 2	5/8	10 ½	0.351
14	320,000	7,630	16.00	14.50	¾	3¾	1.413	1 " " 2	5/8	10	0.368
15	357,000	7,860	17.00	15.50	1	6 ¼	1.510	1 " " 2	5/8	10	0.368
16	394,000	8,160	17.75	16.25	1	6	1.571	1 " " 2	5/8	9¾	0.378
18	474,000	8,630	19.25	17.75	1	5 ½	1.714	1 " " 2	5/8	9 ½	0.388
20	567,000	9,260	21.00	19.50	1	5	1.885	1 " " 2	5/8	9	0.409
22	662,000	9,760	22.50	21.00	1	4 ½	2.090	1 " " 2	5/8	8¾	0.421
24	755,000	10,380	24.00	22.50	1	4 ¼	2.216	1 " " 2	¾	12	0.442
25	830,000	10,750	25.00	23.50	1	4	2.356	1 " " 2	¾	12	0.442

TABLE B - FOR WEARING COAT OVER 3 INS. AND UP TO 9 INS. THICK

CLEAR SPAN	MAXIMUM BENDING MOMENT PER FOOT WIDTH	MAXIMUM SHEAR PER FOOT WIDTH	OVERALL DEPTH OF SLAB EXCLUDING WEARING COAT	EFFECTIVE DEPTH OF SLAB EXCLUDING WEARING COAT	MAIN REINFORCEMENT: DIAM. OF BARS	MAIN REINFORCEMENT: SPACING CENTRES	MAIN REINFORCEMENT: AREA PER FOOT WIDTH AT MIDSPAN	MAIN REINFORCEMENT: NO. OF MAIN BARS CRANKED UP AT EACH END	DISTRIBUTION REINFORCEMENT: DIAM.	DISTRIBUTION REINFORCEMENT: SPACING CENTRES IN THE CENTRAL PORTION OF SPAN	DISTRIBUTION REINFORCEMENT: AREA PER FOOT WIDTH
FT.	INS. LBS.	LBS.	INS.	INS.	INS.	INS.	SQ. INS.	NOS.	INS.	INS.	SQ. IN.
S	M	V	H	d	D		As			e	
3	34,000	3,970	7.00	5.75	3/8	3½	0.408	nil	3/8	7	0.189
4	46,000	4,440	7.00	5.75	½	4¼	0.554	nil	½	9¾	0.241
5	56,000	4,520	8.00	6.75	½	4	0.586	1 in every 4	½	9 ½	0.250
6	102,000	5,670	9.50	8.19	5/8	4½	0.818	1 " " 4	½	7½	0.314
7	121,000	6,050	10.25	8.94	5/8	4¼	0.868	1 " " 4	½	7½	0.314
8	147,000	6,250	11.25	9.94	5/8	3¾	0.982	1 " " 4	½	7½	0.314
9	170,000	6,590	12.00	10.62	¾	5	1.060	1 " " 4	½	7¼	0.325
10	204,000	6,660	13.00	11.62	¾	4¾	1.120	1 " " 4	½	7¼	0.325
12	273,000	7,210	15.00	13.62	¾	4	1,325	1 " " 2	5/8	10½	0.351
14	351,000	7,660	16.75	15.37	¾	3½	1.510	1 " " 2	5/8	9¾	0.378
15	391,000	7,980	17.75	16.25	1	6	1.570	1 " " 2	5/8	9½	0.388
16	437,000	8,310	18.50	17.00	1	5¾	1.640	1 " " 2	5/8	9¼	0.398
18	526,000	8,930	20.25	18.75	1	5¼	1.795	1 " " 2	5/8	9	0.409
20	628,000	9,550	22.00	20.50	1	4¾	1.983	1 " " 2	5/8	8¾	0.421
22	742,000	10,330	23.75	22.25	1	4¼	2.216	1 " " 2	5/8	8½	0.433
24	866,000	11,170	25.50	24.00	1	4	2.356	1 " " 2	¾	11½	0.462
25	935,000	11,310	26.50	25.00	1	3¾	2.510	1 " " 2	¾	11½	0.462

DESIGN LOADING—I.R.C. Standard—Class 'A' (Revised 1947) wheel load train.

DESIGN STRESSES—Concrete. (fc) = 750 lbs. per sq. in; Steel (fs) = 18,000 lbs. per sq. in.

MODULAR RATIO—Fixed 15

Notes:—1. The design tables apply to slabs of bridges & culverts with a clear road width of not less than 2 traffic lanes.

2. Main reinforcement governed by bond and shear considerations up to spans of 4 ft.

3. In table B, the values of Bending Moment for spans upto 5 ft and values of shear for spans upto 12 ft have been kept for convenience the same as in table A, although, for a 9" wearing coat, actual requirements would be somewhat less owing to dispersion effect.

4. The number of main bars to be bent up towards the supports should not exceed that indicated in the tables. The number has been so fixed as to keep bond and tensile stresses on the residual bottom bars within the permissible limits.

5. The area of distribution reinforcement should be not less than $\frac{1}{\sqrt{L}}$ times the area of the main reinforcement. Distribution bars should be securely welded or wired to the main bars at every alternate joint.

6. If full length bars are not available and joints have to be provided, lap joints of length 40 D are permissible provided no joint comes at midspan and all joints are well staggered.

CIRCULAR CULVERTS

CONVEYANCE FACTOR λ.

IN THE FORMULA

$$Q = \lambda\sqrt{2gH}$$

Entry	Diameter Ft ↓ / Length → Ft	20	30	40	50	60	70	80	100	150	200
ENTRY ROUND EDGED	2.0	2.7	2.6	2.5	2.4	2.4	2.3	2.2	2.1	1.9	1.7
	2.5	4.3	4.2	4.1	4.0	3.9	3.8	3.7	3.5	3.2	2.9
	3.0	6.3	6.2	6.0	5.9	5.7	5.6	5.4	5.3	4.8	4·5
	3.5	8.6	8.5	8.3	8.2	8.0	7.8	7.7	7.4	6.9	6.4
	4.0	11.4	11.2	11.0	10.8	10.6	10.5	10.3	10.0	9.3	8.7
	4.5	14.6	14.4	14.1	13.9	13.7	13.5	13.2	12.9	12.1	11.4
	5.0	18.1	17.8	17.6	17.3	17.0	16.8	16.6	16.1	15.2	14.4
ENTRY SHARP EDGED	2.0	2.4	2.4	2.3	2.2	2.2	2.1	2.1	2.0	1.8	1.6
	2.5	3.8	3.7	3.6	3.5	3.5	3.4	3.3	3.2	2.9	2.7
	3.0	5.4	5.4	5.3	5.1	5.1	5.0	4.9	4.7	4.4	4.1
	3.5	7.4	7.3	7.1	7.0	6.9	6.8	6.7	6.5	6.1	5.8
	4.0	9.6	9.5	9.3	9.2	9.1	9.0	8.9	8.7	8.2	7.8
	4.5	12.1	11.9	11.8	11.6	11.5	11.4	11.3	11.0	10.5	10.0
	5.0	14.8	14.6	14.5	14.3	14.2	14.0	13.9	13.6	13.1	12.5

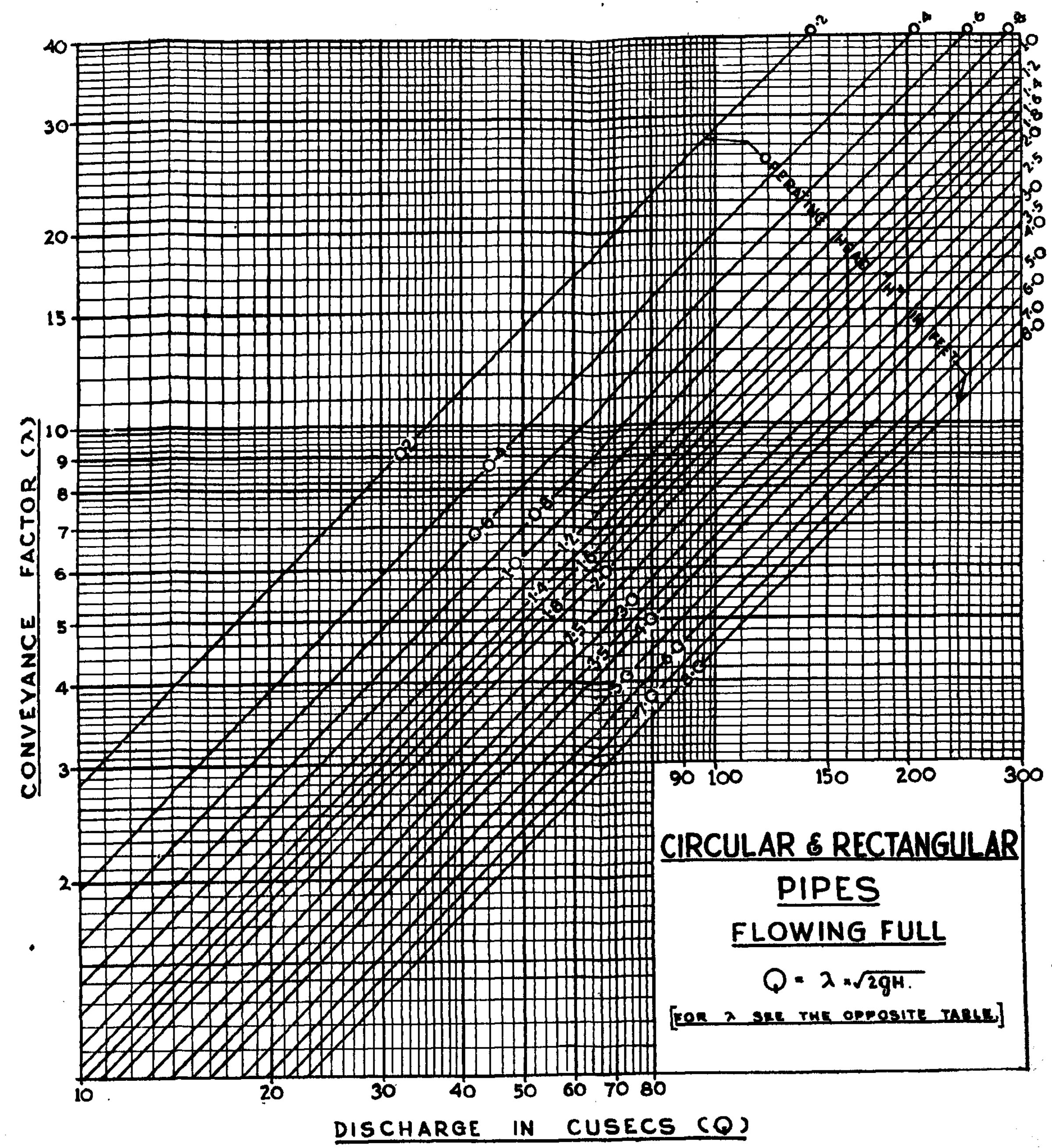
RECTANGULAR CULVERTS

CONVEYANCE FACTOR λ.

IN THE FORMULA

$$Q = \lambda\sqrt{2gH}$$

Entry	Ventway ↓ / Length → Ft	20	30	40	50	60	70	80	100	150	200
ENTRY ROUND EDGED	3 × 2	5.3	5.1	5.0	4.9	4.8	4.7	4.6	4.4	3.9	3.2
	2½ × 2½	5.7	5 5	5.3	5.1	5.0	4.9	4.8	4.6	4.2	3.8
	3 × 3	8.3	8.1	7.9	7.7	7.5	7.4	7.2	6.9	6.3	5.9
	4 × 3	11.1	10.9	10.7	10.5	10.3	10.1	9.9	9.5	8.8	8.2
	4 × 4	15.0	14.7	14.4	14.2	13.9	13.7	13.5	13.0	12.2	11.5
	5 × 4	18.8	18.5	18.2	17.9	17.6	17.4	17.1	16.7	15.6	14.7
	5 × 5	23.6	23.3	23.0	22.6	22.3	22.1	21.9	21.3	20.0	19.0
ENTRY SHARP EDGED	3 × 2	4.9	4.8	4.6	4.5	4.4	4.3	4.2	4.0	3.7	3.4
	2½ × 2½	5.1	5.0	4.8	4.7	4.6	4.5	4.4	4.3	3.9	3.6
	3 × 3	7.4	7.2	7.1	6.9	6.6	6.6	6.5	6.3	5.9	5.5
	4 × 3	9.8	9.7	9.5	9.4	9.2	9.1	8.9	8.7	8.1	7.7
	4 × 4	13.1	12.9	12.7	12.6	12.4	12.2	12.1	11.8	11.1	10.5
	5 × 4	16.4	16.2	16.0	15.8	15.6	15.3	15.0	14.9	14.1	13.8
	5 × 5	20.3	20.2	20.1	19.9	19.8	19.4	19.2	18.8	18.0	17.2



Paper No. 168.

STABILISATION OF CERTAIN CLAYEY SOILS IN INDIA BY HEAT TREATMENT

By

COLONEL S. K. BOSE, B.SC. (ENG.), A.M.I.E. (INDIA), M.R.S.I. (LONDON).

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SYNOPSIS

This paper presents the results of a laboratory study on the stabilization of the black cotton soils (Montmorillonite group) found in the Deccan. The application of this method of stabilization on certain alluvial soils in the Gangetic plain is also indicated.

Black cotton soil could not be stabilized even with 20 per cent of cement but a minimum of 10 per cent cement with 10 per cent lime promised a satisfactory result. The same strength can also be obtained by the addition of 40 to 50 per cent sand to the soil before mixing with 10 per cent cement. But none of these methods are likely to be economical.

Heating this soil to 500 deg. C. brings it virtually to the nature of a sandy soil which can be stabilized with 10 per cent cement. But the addition of 20 per cent natural soil to the heat-treated soil before the addition of 10 per cent cement greatly increases its compressive strength and brings it to a value equal to that obtained by mixing with 10 per cent cement and 10 per cent lime or with 40 to 50 per cent sand and 10 per cent cement.

This soil can also be stabilized with the addition of 7½ per cent natural soil to the heat-treated soil and 6 per cent road tar No. 3.

These methods were applied to certain alluvial soils from Bengal (both Montmorillonite and Kaolinite groups) and the results are given. The action of lime on these soils is not as significant as in the case of the black cotton soils. The addition of natural soil to the heat-treated soil of Kaolinite group shows decrease in strength against the increase noticed in black cotton soils. However, the effect of heat-treatment on these two soils from Bengal although not as remarkable as in the black cotton soil is still noticeable.

It is considered that a stage in the laboratory investigation has reached when confirmatory data from field trials are essential to indicate further investigation.

INTRODUCTION

1. In India there is lack of progress in any large-scale soil-stabilized construction of roads or airfields in spite of the the recent advances both in soil science and in the technique of construction. This may be attributed, to a large extent, to the peculiar soil-types which are not economically amenable to stabilization for large-scale construction.

2. Principal Soil Types. Pedologically, India may be divided broadly into three main regions namely :

(a) The Peninsula covered partly by red soils and laterite soils and partly by black cotton soils.

(b) The Gangetic Plain containing alluvium in various degrees of maturity.

(c) The Himalayan highlands having marked vertical zonation and including brown earths developed naturally under deciduous forests, pedsolized soils and clayey soils.[1]*

3. The soil map of India by Dr. J. K. Basu[2] Figure 1, shows the incidence of the principal soil-types. It is seen that black cotton and alluvial soils cover more than half of the country.

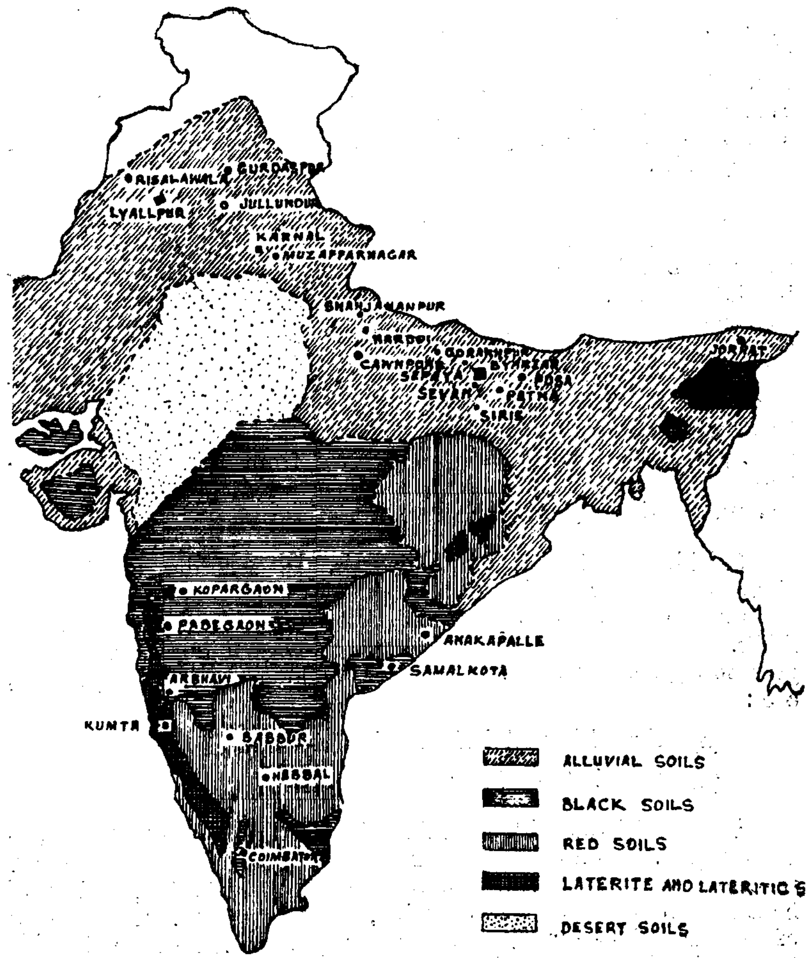


Fig. 1

Soil Map.

*Refer to serial No. in the bibliography.

4. As far as is known, a suitable and economical method of large-scale road construction by stabilization of black cotton soils found in India has not yet been developed. Investigations were, therefore, initiated by the Author at the Engineering Research Wing of the College of Military Engineering, Kirkee, on a laboratory scale in 1951. This investigation has now reached a stage where field trials are desirable.

The laboratory method of stabilizing black cotton soil has also been applied to the stabilization of certain alluvial soils of the Gangetic Plain. Because of the varieties of alluvial soils found in this region, the latter study is not yet complete. However results so far obtained are included in this Paper in order to assess the applicability or otherwise of this method to other clayey soils.

5. Heat Treatment of Clay. The burning of clayey soils into bricks as a means of stabilization is well known. But the temperature of burning to get good bricks is very high, often approaching 1200 deg. C. It has been found in the laboratory study that at a temperature approximately half of the brick-burning temperature, certain changes in the physical constituents of clayey soils take place which make the stabilization of these soils comparatively easy. The present investigation shows that the heat treatment cannot be applied indiscriminately to any type of clayey soil because the benefits derived from it vary with soil-types. The treatment is likely to be economical only for soils having certain clay mineral characteristics.

6. Identification of Clay Minerals. In accordance with the character of their lattice structures, clay minerals are normally classified into three groups, viz., *Kaolinite, Montmorillonite and Illite.* The methods of identification of clay mineral, which also give an approximate estimate of the amount of minerals present, are :

(a) the Differential Thermal analysis, and

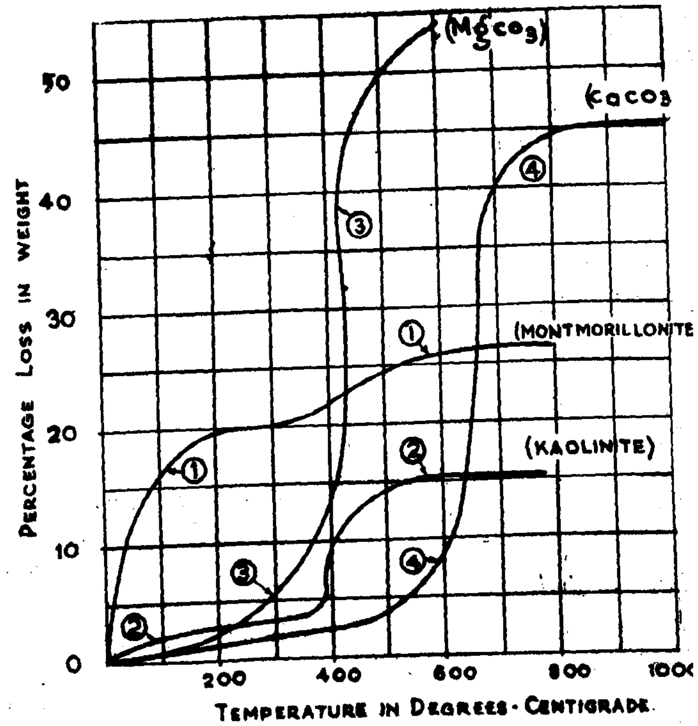
(b) the X-Ray Diffraction analysis.

In the Differential Thermal analysis, the sample is heated to 1000 deg. C. at a uniformly increasing rate in the course of which reactions occur, which in some cases exothermic i.e., giving off heat and in others endothermic i.e., absorbing heat. The pattern of these reactions for a particular soil type provides a good guide to the identification of the mineral contents and estimating their quantities.

The X-Ray analysis is based on the principle that crystal-planes diffract X-Rays. The diffracted X-rays are registered on a

photographic film and the dimensional relationships of these lines are used as a basis of computing the plane-spacing of crystal planes[3].

In this investigation, the identification of clay minerals was based on dehydration curves. This method is dependent on the loss in weight of samples heated to different temperatures. The loss in weight is plotted against the temperature and the characteristic dehydration curves are obtained. The dehydration curves obtained by Kelly, Jenny and Brown[4] are shown in figure 2. Dehydration curves of certain Indian soils published by Seshadry and Krishnamurthy[5] have also been used for comparison.



1 and 2 After Kelly, Jenny and Brown
3 After Mellor
4 After Seshadry and Krishnamurthy

Fig. 2

Typical Dehydration Curves.

EXPERIMENTS ON BLACK COTTON SOIL

7. **The Black Cotton Soils of India.** The black cotton soil, in India, is a heavy textured clayey soil derived from Deccan trap which is a black basalt. The organic content is generally low and the black colour is generally attributed to the presence of finely divided tetaniferous magnetic. The soil generally occurs in regions where the rainfall is insufficient to compensate for the evaporation resulting from high temperatures and thus incomplete leaching results. The soil is characterised by a zone of calcium carbonate concretions (Kankar). The depth of the soil varies from 3 to 6 feet, occasionally going down 30 to 40 feet. On drying, the soil shrinks and cracks to such an extent that it is often said that "the black cotton soil ploughs itself"[1]. The soil used in the investigation was obtained from the neighbourhood of Poona.

8. **Characteristics of the Black Cotton Soils.** Characteristics of the black cotton soil investigated are given below :

TABLE I

CHARACTERISTICS OF BLACK COTTON SOIL

(a) *Particles Size Analysis.*	per cent.		
Gravel ...	1.0	Specific gravity	2.76
Coarse sand ...	4.2		
Fine sand ...	24.0	CBR value (modified AASHO at saturate condition)	2.3 per cent.
Silt ...	16.8		
Clay ...	54.0		
(b) *Plasticity limits.*			
Liquid limit ...	62.5	Maximum density AASHO	103 lbs. per cu. ft.
Plastic limit ...	34.2		
Plasticity index ...	30.1		
(c) *Shrinkage tests.*			
Shrinkage limit ...	17.80	Optimum Moisture content	24.2 per cent.
Shrinkage ratio ...	2.12		
Linear shrinkage ...	17.50	pH	7·3

(d) *Chemical Characteristics.*

	Per cent
Residue of soil samples passing through No. 40 sieve remaining insoluble in hydrochloric acid	59.33
Fe_2O_3	9.29
Al_2O_3	13.68
Ca 0	6.11
Mg 0	3.98
Organic matter	1.53

The dehydration curve of the black cotton soils tested is shown in figure 3. The curve was obtained as described in para 6.

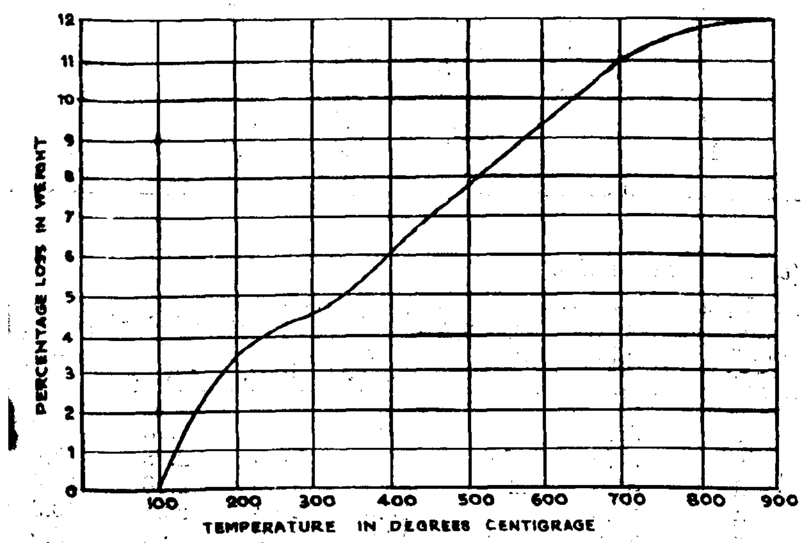


Fig. 3

Dehydration Curve of the Black Cotton Soils Investigated.

9. **Granular Stabilization of the Black Cotton Soil.** Soil samples in which the proportion of clay (less than 0.002 mm. dia.) ranged from 42.4 to 1.2 per cent were prepared by the addition of varying quantity of sand. The physical properties indicated that a mix of 2 to 3 parts of black cotton soil and 7 to 8 parts of sand gave the minimum plasticity index, minimum linear shrinkage, maximum shrinkage limit and maximum dry density.

TABLE 2

PROPERTIES OF VARIOUS MIXES OF BLACK COTTON SOIL AND SAND

(a) *Mechanical Analysis.*

Soil sample No.	Gravel	Coarse sand	Fine sand	Silt	Clay.
1	1.7	9.0	29.9	17.0	42.4
2	0.7	16.9	26.4	16.0	40.0
3	2.1	29.0	18.7	13.8	36.4
4	3.4	37.1	10.9	15.2	33.4
5	4.4	42.0	14.6	9.2	19.8
6	2.8	50.9	12.5	8.0	25.8
7	5.8	54.2	16.4	8.6	14.8
8	5.3	66.9	11.8	6.7	9.3
9	6.8	78.7	5.1	3.9	5.5
10	8.3	83.9	5.4	1.2	1.2

(b) *Index Properties.*

Soil Sample No.	Liquid limit.	Plastic limit.	Plasticity index.	Shrinkage limit.	Shrinkage ratio.	Linear limit.
1	56.0	29.8	26·2	17.1	2.15	17.5
2	51.0	28.0	23.0	15.8	2.08	15.9
3	41.0	22.5	18.5	15.4	2.06	14.4
4	36.0	20·2	15.8	16.4	2.03	10.0
5	34.8	18.8	16.0	18 9	1.98	8.0
6	33.1	18.2	14.9	19.2	1.95	8.0
7	31.5	18.1	13.4	18.5	1.90	5.6

Samples 8, 9 and 10 showed very little change from sample No. 7

The results in Table 2 indicate that black cotton soil can be stabilized only by the addition of coarse sand 3 to 4 times its bulk. As sand has to be transported over long distances in the Poona region, granular stabilisation of black cotton soil is uneconomical.

10. Stabilisation with Bituminous Road Oil. The soil mixes shown in Table 2 were tried with different percentages of shell soil-stabilisation oil (SSS Oil) which is a commercial product, containing bitumen, fuel oil and wax. This oil, ranging from 1 to 4 per cent by weight of soil, was used alone as well as mixed with 2 to 3 per cent of lime. Specimens were subjected to direct shear, crushing, cone-penetration and water absorption tests. The test results clearly indicated that the oil did not give satisfactory stabilization of the black cotton soil.

These tests also showed:

(a) Although the shear strength of SSS oiltreated soil was greater than that of the untreated soil, the soil sample became saturated when immersed in water, indicating that the treatment by SSS oil failed to waterproof the soil particles.

(b) The shear strength of the saturated sample of the black cotton soil treated with SSS oil was higher when mixed with sand than without the sand. The addition of sand, therefore, improved the water-proofing quality of SSS oil.

(c) The addition of lime did not improve the water-proofing action of the oil to any appreciable extent.

11. Effect of Chemical Stabilizers.

The following wellknown chemical stabilizers were also tried: –

(a) Aneline–Furfural,
(b) Calcium Stearate,
(c) Sodium Rosinate,
(d) Cut-back bitumen RC + Aluminium Soap, and
(e) Trethanol-Amine.

Samples were prepared with various chemical stabilizers noted above and were subjected to crushing and water absorption tests. Aneline—Furfural is the most effective of the stabilizers used. But none of the chemical stabilizers was very satisfactory.

12. Stabilization with Cement. Varying percentages of cement were added to the various mixes of soil and sand indicated in Table 2 to determine the stabilizing effect. The test specimens were subjected to crushing and durability tests, i.e., 12 cycles of alternate wetting and drying. Figure 4 shows the compressive strength of black cotton soil when mixed with various proportions of sand and cement, while figure 5 shows the index limits of the soil with various percentages of cement. The composition of the mixture of sand and soil used was the same as soil sample No. 10 in Table 2.

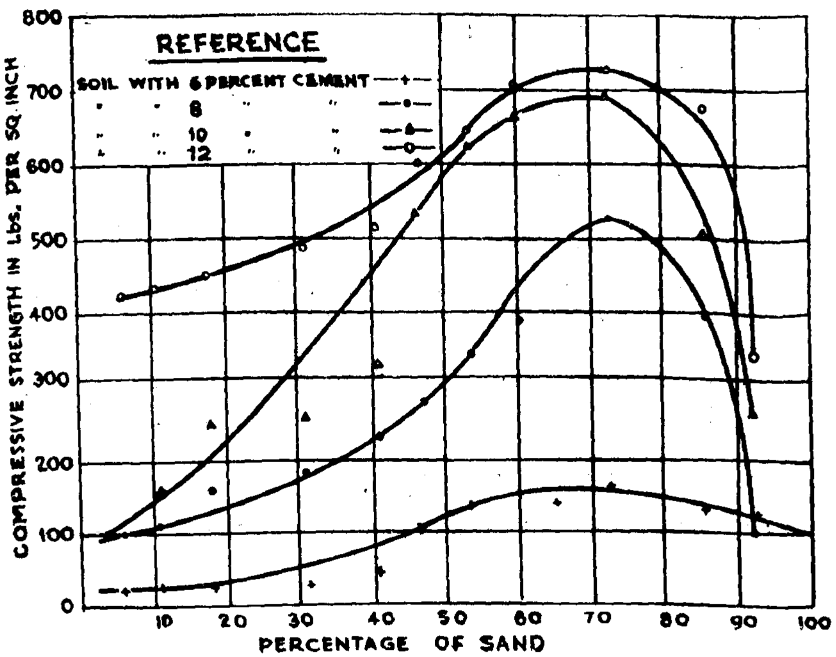


Fig 4.

Compressive strength of Black Cotton Soil when mixed with different proportions of sand and Cement.

Figure 4 indicates that the maximum compressive strength generally occurs when 70 per cent sand and 30 per cent black cotton soil is mixed with cement.

Also 8 per cent cement plus 45 per cent sand or 10 per cent cement plus 25 per cent sand mixed with the soil gave compressive strength of 250 lb per sq. in. The large quantity of sand required for stabilization with cement precludes the use of this method in

areas where sand is not available in large quantities within economic haulage.

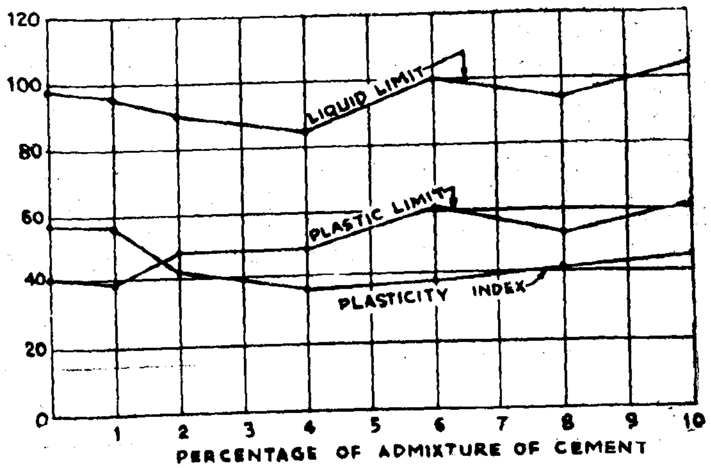


Fig 5.

Index Limits of Black Cotton Soil mixed with different percentages of Cement.

Without the admixture of sand the black cotton soil in its natural state could not be stabilized satisfactorily with even 20 per cent cement as shown in Table 3.

TABLE 3

THE COMPRESSIVE STRENGTH OF SOIL-CEMENT MIXES AFTER 7 DAYS CURING.

	lbs per sq. in.
Black cotton soil+10 per cent cement	102
Black cotton soil+15 per cent cement	145
Black cotton soil+20 per cent cement	192

The results obtained were similar to the results obtained when lime was used as a stabilizer. See para 13.

13. **Stabilization with lime.** The determination of exchangeable bases in black cotton soils is complicated by the presence of considerable amounts of free calcium carbonate. From the

results of analysis published by Basu[2] it is seen that the principal base is calcium accompanied by small amounts of magnesium, potassium and sodium. The total base saturation capacity, as found by Basu, in the upper layers of black cotton soils in the Deccan canal areas varies from 65 to 78 per cent. Addition of lime to the natural black cotton soils was found to improve their physical properties but the actual reaction of lime is still obscure. Lime helps the coagulation of suspended matter thereby reducing the clay particle distribution in the sample.

The effect of mixing varying percentages of lime with the black cotton soil in changing the index limits is shown in figure 6. Although Liquid Limits decreases with the increase in the proportion of lime, the Plastic Limit increases at first and remains high.

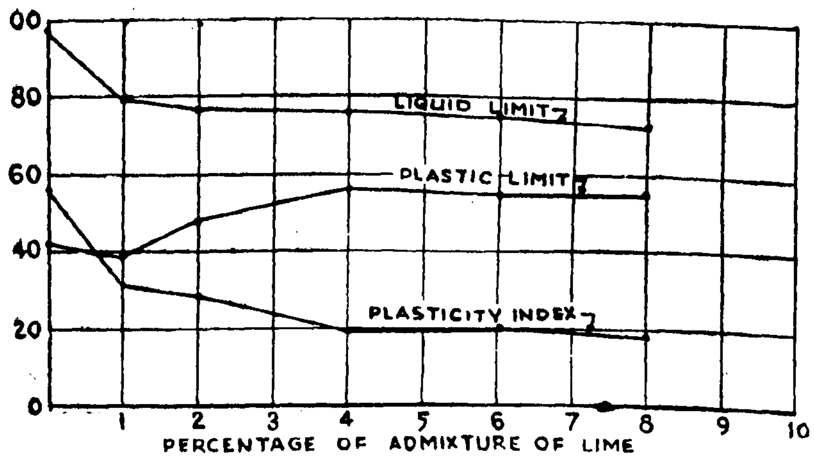


Fig 6.

Index Limits of the Black Cotton Soil mixed with different percentages of lime.

It was also observed that the addition of 4 per cent or more of lime made the effective dispersal of the suspension difficult for the determination of silt and clay. These samples (with lime) were cured for 48 hours before carrying out the tests.

That the addition of lime or cement decreases the Liquid Limit and increases the Plastic-Limit was also reported by Spangler and Patel[9] in a laboratory study of the engineering properties of a heavy, sticky, clayey soil in Southwest Iowa, U. S. A. This soil was selected by them because outwardly it appeared to be similar to black cotton soil in Southern India. The results published in the paper referred to above are reproduced below:

TABLE 4

RESULTS OF PLASTICITY TESTS ON GUMBO SOIL AND VARIOUS MIXES OF LIME AND CEMENT

Mixture	Liquid Limit	Plastic Limit	Plasticity Index
Natural soil	69.0	27.3	41.7
Soil-lime Mixtures			
Soil + lime 1 per cent	64.0	47.0	17.0
Soil + lime 2 per cent	61.5	49.5	12.5
Soil + lime 3 per cent	57.0	46.0	11.0
Soil + lime 4 per cent	54.5	44.5	10.0
Soil + lime 8 per cent	51.0	43.5	7.5
Soil-cement Mixtures			
Soil + cement 1 per cent	62.0	32.0	30.0
Soil + cement 2 per cent	60.0	38.0	20.0
Soil + cement 3 per cent	58.0	36.0	22.0
Soil + cement 4 per cent	55.0	35.0	20.0
Soil + cement 8 per cent	55.0	47.0	7.5

14. Stabilization with Cement and Lime. The results shown in Figs. 5 and 6 led to the use of cement and lime combination as a possible stabilizing agent. Different percentage combinations of lime and cement were mixed with the black cotton soil and the specimens were subjected to tests after curing for 7 days. The results are shown in Table 5.

TABLE 5—TABLE SHOWING THE IMPROVEMENT IN STRENGTH AND WATER-PROOFING ACTION OF CEMENT-LIME STABILIZATION

(a) *Crushing Strength*

Percentage of stabilizing agents		Crushing strength after 7 days curing & immersion in water for 48 hours
Lime	Cement	lb per sq. inch
3	6	53
6	3	69
3	10	120
6	10	172
10	3	227
10	6	276
10	10	448

(b) *Swelling and C.B.R. Values*

Percentage of stabilizing agents		Percentage of swelling after 4 days immersion	C. B. R. value after 4 days immersion
Lime	Cement		
0	0	7.4	1.9
3	3	0·12	142
3	6	0·05	208
6	3	1·50	203
6	6	1·50	250
10	3	0·00	250
10	6	0·00	250
10	10	0·00	250

Note : The cement and lime formed a structural material after curing and the C.B.R. values were obtained from the maximum reading of the penetration dial gauge. This usually occurred before the piston could penetrate for 0.1 inch because crushing took place before penetration was achieved.

The results shown in Table 5 indicate that a combination of at least 10 per cent lime and 10 per cent cement might give satisfactory stabilization of the black cotton soil. This can only be confirmed by field trials.

The results with 10 per cent lime and 10 per cent cement are better than with 20 per cent cement (see Table 3 page 225). The practical application of this method in areas far from railways is questionable because of the costs involved in the transport of cement and lime.

15. **Effect of Heat-treatment of the Soil.** Experiments were conducted to see the effect of heat on the physical properties of the black cotton soil. The experiments were prompted by the expectation that at high temperatures, the crystalline lattice structure of the clay mineral might be disrupted due to the loss of water molecules and that free alumina and silica liberated in the process might form complex Aluminosilicates with further rise of temperature. The physical properties may be expected to depend on the duration and temperature of the heat-treatment and the nature of the clay minerals.

The black cotton soil was heated to different temperatures in an electric muffle furnace and its physical properties were examined to see if the heat treated soil was amenable to stabilization economically with stabilizing agents such as cement or bitumen.

The physical properties of black cotton soil heated to different temperatures are given in Table 6.

TABLE 6

PHYSICAL PROPERTIES OF BLACK COTTON SOIL HEATED TO DIFFERENT TEMPERATURES

(a) Index Properties of the heated soil

Temperature maintained for one hour	Liquid Limit	Plastic Limit	Plasticity Index
degrees Centigrade			
300	52.3	32.1	20.2
340	47.7	31.3	16.4
360	45.4	32.0	13.4
400	—	—	Non-plastic
500	—	—	do
600	—	—	do
800	—	—	do

TABLE 6 *(Continued)*

PHYSICAL PROPERTIES OF BLACK COTTON SOIL HEATED TO DIFFERENT TEMPERATURS

(b) Mechanical Analysis of the Heated Soil

Temperature of heating for one hour	Coarse sand	Fine sand	Silt	Clay
Degrees Centigrade	per cent	per cent	per cent	per cent
300	4.2	24.5	17.6	53.7
340	4.7	26.5	23.1	45.7
400	5.0	26.8	25.2	43.0
500	60.8	21.4	11.9	5.9
800	88.0	10.5	0.1	1.4

(c) Shear Value of the Heated Soil

Temperature of heat ing for one hour	Apparatus used	Angle of internal friction	Cohesion
Deg. Centigrade			
500	Constant strain Disc shear apparatus	42°	1 lb per sq. inch
500	Triaxical shear apparatus	41.2°	2 lb per sq. inch

It will be observed from the tabulated results that the black cotton soil has virtually the consistency of sand when subjected to a temperature of about 500 deg. C. The change in the soil structure that takes place when the temperature is raised from 400 deg. C. to 500 deg. C. is remarkable (see Table 6 – Mechanical analysis). This observation is corroborated by Grainger in his laboratory study on a sample of black cotton soil obtained from Nigeria.

The observation recorded by Grainger on Nigerian soil is interesting. In his tests for producing a low-grade aggregate from black cotton soil, the best results were obtained by heating soil-samples to temperatures ranging between 500 deg. C. and 700 deg. C. At lower temperatures, the material still possessed the properties of a soil and was easily broken up on impact. When heated to over 700 deg. C. the material became harder and more brittle till at the higher temperatures of burning it was again more easily shattered.

16. **Stabilization of Heat Treated Soil with Cement.** The first set of laboratory investigations were carried out to see if the black cotton soil heated to 500 deg. C. could be stabilized with cement. The average crushing strength of soil samples stabilized with 10 per cent cement at the optimum moisture content was found to be:

	Crushing strength lb per sq. inch
(a) without curing	22
(b) After curing for 7 days	126

As these values were much lower than the acceptable minimum value of the strength of cement-stabilized soil samples, further tests were carried out in which the black cotton soil, heated to 500 deg. C. for one hour, was mixed with different percentages of natural black cotton soil and then stabilized with 10 per cent of cement. The results on Proctor compacted specimens after 7 days of damp curing are shown in Table 7.

TABLE 7

COMPRESSIVE STRENGTHS OF SPECIMENS OF HEATED BLACK COTTON SOIL (500° C) STABILIZED WITH 10 PER CENT CEMENT AND DIFFERENT PERCENTAGES OF NATURAL BLACK COTTON SOIL

Percentage of natural black cotton soil added to the heated soil. (Cement 10 per cent throughout)	Compressive strength in lb per sq. inch	Remarks
0.0	126	
7.5	293	
12.5	380	The results tabulated are the average of six samples.
20.0	420	
25.0	356	
30.0	214	

On the other hand, if lime is added to the mix of heated black cotton soil with 10 per cent cement, the increase in the compressive strength of the soil sample registers no improvement on the strength obtained by mixing natural soil as will be seen from the following Table.

TABLE 8

COMPRESSIVE STRENGTH OF SPECIMENS OF HEATED BLACK COTTON SOIL (500° C) STABILIZED WITH 10 PER CENT CEMENT AND VARIOUS PROPORTIONS OF LIME

Percentage of lime	Crushing strength lb per sq. inch
0	126
2	165
4	217
6	277

The results shown in Table 7 are interesting. The compressive strength of heated soil-cement mix increased with the addition of natural soil up to 20 per cent. Beyond this limit, further addition of the natural soil decreased the compressive strength.

The optimum content in increasing the compressive strength of the stabilized soil is understandable. Shear resistance is one of the important physical properties for soil stability.

Shear resistance = Cohesion plus normal load × coefficient of internal friction.

Until there is enough clay to coat the non-clay minerals and supply appreciable cohesion, maximum shear resistance of non-plastic soils is governed by friction, cohesion being negligible. As the clay content increases so as to coat all non-clay mineral surfaces, cohesion becomes more important and is maximum when all the individual clay surfaces are separated by water films of a definite thickness. There will be a decrease in cohesion if the thickness of water films is greater or less than the critical thickness[8]. As the soil in its natural state of moisture content was mixed with the heated soil the shear resistance and hence the compressive strength of the mix was reduced when the clay content increased beyond the amount required to coat all non-clay mineral surfaces.

That the addition of natural soil to the heated soil in increasing the strength of the soil-cement mix is advantageous as it should reduce the cost of construction considerably.

17. Stabilisation of the Heated Soil with Road Tar No. 3. The heated black cotton soil when treated with Road Tar No. 3 gave very promising results. It was found that the minimum quantity of the road tar required for a thorough mixing with the black cotton soil in its natural state was 10 per cent.

But the proportion of tar can be reduced by heat-treating the soil. Small quantities of the black cotton soil heated to 500 deg. C. and treated with 5 per cent road tar, when immersed in water for over a month, did not show signs of disintegration.

Further tests on the heated soil mixed with different percentages of natural black cotton soil and 6 per cent of road tar were carried out. The results obtained are shown in Table 9. Although 5 per cent tar could be mixed with heated soil alone, it was found that at least 6 per cent tar was required for mixing when natural soil had been added to the heated soil.

TABLE 9

COMPRESSIVE STRENGTH OF SPECIMENS OF HEATED BLACK COTTON SOIL (500° C.) STABILIZED WITH 6 PER CENT ROAD TAR NO. 3 AND DIFFERENT PERCENTAGES OF UNHEATED BLACK COTTON SOIL (NATURAL SOIL)

After seven days air curing of Proctor compacted specimens

Percentage of unheated black cotton soil	Compressive strength in lb per sq. inch.	
0.0	112	(5 per cent Road Tar used in this case.)
7.5	329	(6 per cent Road Tar was used.)
10.0	280	
12.5	270	
15.0	267	
20.0	267	

It is observed that for stabilization of heated black cotton soil with Road Tar No. 3, the optimum clay content is obtained by the addition of 7.5 per cent of natural soil. A fairly reasonable strength, 329 lb per sq. in. is obtained.

18. Summary. The black cotton soil used in this investigation has the following characteristics :

	Per cent
Clay	54.0
Silt	16.8
Fine sand	4.2
Gravel	1.0
Liquid limit	62.5
Plastic limit	32.4

The Granular stabilization of the black cotton soil requires the addition of coarse sand 3 to 4 times the quantity of the soil.

The soil could not be satisfactorily stabilized with bituminous road oil and the latter failed to waterproof the soil particles effectively even in combination with lime. Also, the addition of various chemical stabilizers did not give satisfactory result.

The black cotton soil in its natural state could not be stabilized with even 20 per cent cement. This soil when mixed with 70 per cent of sand and 10 per cent cement gave satisfactory results. The addition of lime improved stabilization. A minimum of 10 per cent cement with 10 per cent lime gave better results than with 20 per cent cement only.

But none of these methods of stabilization can be considered economical enough to be used in road or airfield construction.

Heat-treated black cotton soil, on the other hand, can be stabilized with cement or Road Tar No. 3 provided it (heat-treated soil) is mixed with the requisite quantities of natural black cotton soil to provide the necessary finer particles and cohesion. Heat-treated soil mixed with 20 per cent natural soil and 10 per cent cement gives the best results. When Road Tar is used, only 7.5 per cent of natural soil plus 6 per cent Road Tar No. 3 give the maximum strength. All percentages mentioned are by weight of the heat treated soil.

INVESTIGATION RELATING TO ALLUVIAL SOILS IN GANGETIC PLAIN

19. Two representative samples of alluvial soils of the Gangetic plain were obtained for investigation from the deposits in the humid climate of Bengal.

(1) From Kankinara which is situated about 12 miles north of Calcutta, and

(2) from Panagarh which is situated about 100 miles north-west of Calcutta.

Both groups of alluvial soils show high silt contents.

20. Characteristics of Alluvial Soils. Characteristics of the two soil types are given in Table 10.

TABLE 10

CHARACTERISTICS OF ALLUVIAL SOILS

		Panagarh soil	Kankinara soil
(a) *Particle size analysis*			
Coarse sand	Per cent	14.9	1.6
Fine sand	,,	28.9	31.7
Silt	,,	37.1	38.8
Clay	,,	19.1	27.9
Optimum moisture content	,,	13.1	15.0
Density		121 lb per cu. ft.	112.2 lb per cu. ft.
(b) *Plasticity Indices*			
Liquid limit		32.8	48.6
Plastic limit		17.4	27.2
Plasticity index		15.4	21.4
(c) *Shrinkage*			
Shrinkage limit		17.2	21.4
Shrinkage ratio		1.8	1.804
(d) pH ...		6.3	7.1

The dehydration curves obtained for Kankinara and

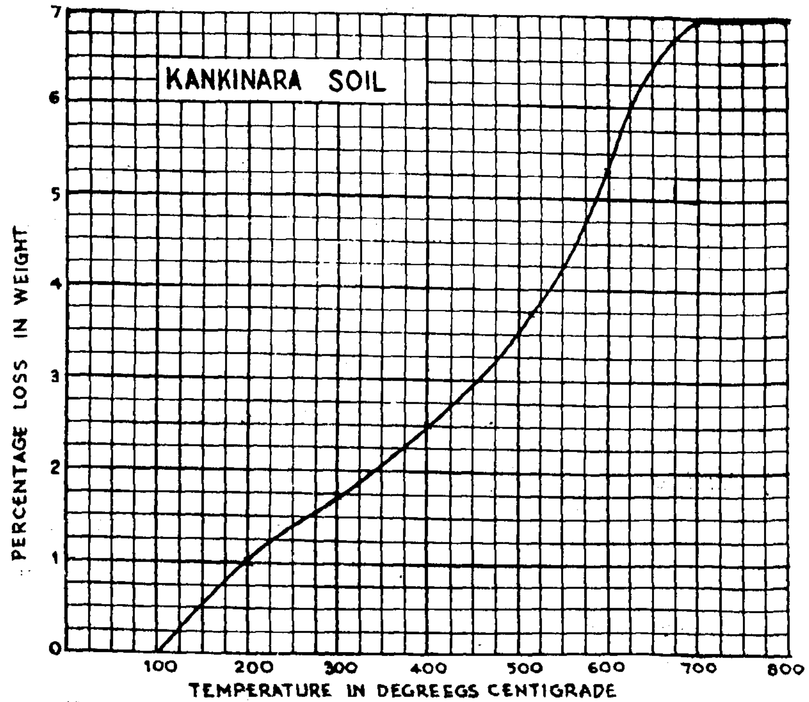


Fig. 7 Dehydration Curve of Kankinara Soil.

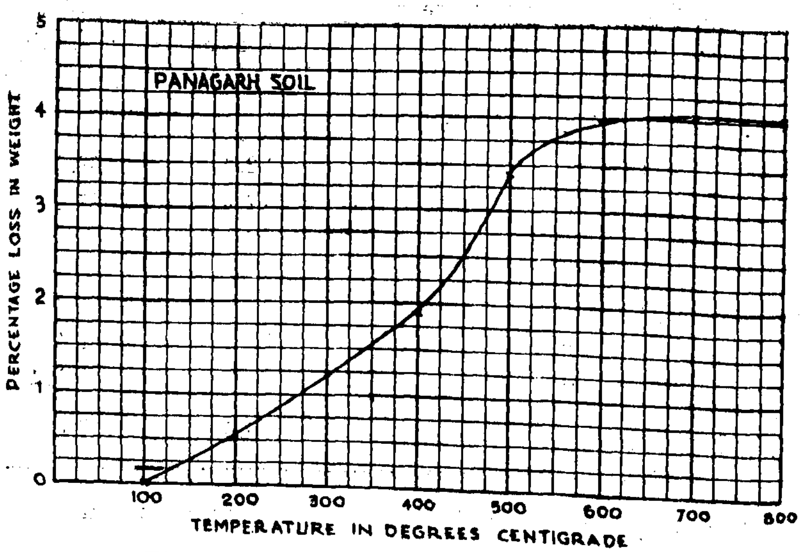


Fig. 8 Dehydration Curve of Panagarh Soil.

Panagarh soils are shown in Figs. 7 and 8 respectively.

The dehydration curve of Kankinara soil, Fig. 7, shows that the soil type is more akin to Kaolinite group than to Montmorillonite group as the sharp rise in the loss in weight due to dehydration occurred in the temperature range of 550 deg. C. to 650 deg. C.

The dehydration curve of Panagarh soil, Fig. 8, shows that the soil type is akin to the Montomorillonite group as the sharp rise in the loss of weight due to dehydration occurs over a temperature range of 400 deg. C. to 550 deg. C. The soil from Kankinara is therefore likely to behave more like black cotton soil than the soil from Panagarh.

21. The investigation is designed to show the comparative effect on the two soil types using the mixes that were found satisfactory with the black cotton soil. These results are more in the nature of showing the limitations in the application of results obtained with one soil to another rather than discovering the best method of stabilizing the two soil types. The investigation also brings out the desirability of confirming heat-treatment of soil to certain types only for the purpose of stabilization.

22. **Stabilization with Cement and Lime.** The action of lime in increasing the compressive strength of soils from Kankinara and Panagarh was first studied. The results are shown in Table 11.

TABLE 11

EFFECT OF LIME ON THE COMPRESSIVE STRENGTH

Percentage of lime added to natural soil	Compressive strength after 7 days curing	
	Panagarh soil	Kankinara soil
	lb. per sq. in.	lb. per sq. in.
2	81	42
4	99	78
6	111	96

The general conclusion is that the effect of lime in increasing the compressive strength of soil is more pronounced in Montmorillonite group than in Kaolinite group. It is also observed that the action of lime is more pronounced on the black cotton soil than on the Kankinara soil. Further observations made in stabilizing these two soil-types with a mixture of cement and lime confirm the observations made with lime as admixture to the soils.

Table 12 shows the compressive strength of specimens of unheated soils with various mixes of cement and lime.

TABLE 12

COMPRESSIVE STRENGTH OF UNHEATED SOILS WITH VARIOUS MIXES OF CEMENT AND LIME

Percentage addition of cement and lime to natural soil		Compressive strength after 7 days damp curing on Proctor compacted specimens	
Cement	Lime	Panagarh soil	Kankinara soil
		lb. per sq. in.	lb. per sq. in.
5	0	195	155
10	0	370	221
10	2	384	236
10	4	384	244
10	6	384	251
15	0	550	478
15	2	550	485

The addition of 2 per cent of lime to Panagarh soil does not give results commensurate with cost. Further addition of lime to the soil has no effect in increasing the compressive strength of the mix.

Although the addition of lime to Kankinara soil shows some improvement in the compressive strength of the mix, the result is not as significant as in the black cotton soils. Moreover, addition of 10 per cent cement and 6 per cent lime gives a compressive strength of 251 pounds per sq. inch whereas 15 per cent cement (which will be equivalent in cost to 10 per cent cement and 6 per cent lime) gives the much higher value of 478 pounds per sq. inch.

23. Heat-treatment. The physical properties of the two soil types after these are heated to 500 deg. C. are shown in Table 13.

TABLE 13

MECHANICAL ANALYSIS OF THE SOIL-TYPES AFTER HEATING TO 500 DEG. C. FOR ONE HOUR

Particle-size analysis	Panagarh soil	Kankinara soil
	Per cent	Per cent
Coarse sand ...	28.1	27.3
Fine sand ...	35.6	25.2
Silt ...	32.6	39.3
Clay ...	3.7	8.2

TABLE 14

COMPRESSIVE STRENGTH OF HEAT-TREATED SOILS WITH VARIOUS MIXES OF CEMENT AND UNHEATED SOIL

Percentages of additions to the heat-treated soil	Compressive strength in lb. per sq. inch after 7 days damp curing on Proctor compacted specimens	
	Panagarh soil	Kankinara soil
5 cement only	94	89
5 cement plus 20 natural soil	83	170
7·5 cement only	361	256
7·5 cement plus 20 natural soil	318	275
10 cement only	516	372
10 cement plus 20 natural soil	413	381

The results shown in Table 14 reveal that the addition of 20 per cent natural soil to heated Panagarh soil reduces the compressive strength rather than increase it. On the other hand, the addition of 20 per cent natural soil to Kankinara soil improves the compressive strength slightly. The black cotton soil, however, showed a remarkable increase in compressive strength with the addition of natural clay up to 20 per cent.

A low cement-soil mix (5 per cent cement) reduces the compressive strength of the mix more in the heated soil than in the case of unheated soil. The natural soils with 5 per cent cement gave compressive strengths of 195 and 155 pounds per sq. inch for the Panagarh and Kankinara soils respectively, whereas these values were reduced to 94 and 89 lb per sq. inch respectively when 5 per cent cement was added to heat-treated soils. (See Tables 12 and 14).

24. Stabilization using Road Tar with or without heat-treatment. Road Tar No. 3 was mixed hot with the soil and the samples were cured for 7 days in the air before testing for compression. In the natural state, the soils required 10 per cent Road Tar for thorough mixing. The compressive strengths obtained are shown below :

TABLE 15

COMPRESSIVE STRENGTH OF NATURAL SOIL WITH ROAD TAR

Percentage of Road Tar used	Compressive strength	
	Panagarh soil	Kankinara soil
	lb per sq. in.	lb per sq. in.
10	384	192
12	414	222

When Road Tar was mixed with heat-treated soil (without the addition of natural soil to the heated soil), it was found that a minimum of 7 per cent of tar was required for thorough mixing with Kankinara soil and a minimum of 10 per cent with Panagarh soil. The results are given below :

TABLE 16

COMPRESSIVE STRENGTH OF HEAT TREATED SOIL

Soil type	Percentage of tar	Compressive strength
		lb per sq. in.
Kankinara soil	7	258
Panagarh soil	10	458

The effect of the heat-treatment for stabilization of soil could be seen from the foregoing. But the method of stabilization must be varied to suit different soils. For example, the addition of natural soil up to 20 per cent is found necessary in the case of black cotton soil heated to 500 deg. C. whereas any addition of natural soil to heated Panagarh soil results in decreasing its compressive strength.

METHOD OF APPLICATION OF HEAT TREATMENT ON A LARGE SCALE

25. The method of heat treatment of soil used in Australia is mechanical and is briefly described below.

It is found most useful for heavy clayey soils which break up in wet weather. Clay soils containing substances such as lime or potash are most easily treated ; refractory brickmaking clay needs higher temperature.

The road surface is prepared for burning by forming and well consolidating either by watering and rolling or by traffic under wet weather conditions. For satisfactory results it is essential that the compaction is complete before burning. This can only be done by thoroughly wetting the earth after formation.

The machine consists of a travelling reverberatory type of furnace which burns firewood, the latest model being about 33 ft long overall with the reverberatory portion of the furnace about 20 ft long and 6 ft wide ; the level of the furnace is adjustable. The whole machine weighs about 26–27 tons and travels on its own road wheels. A petrol engine and gearing are fitted for the purposes of locomotion. It can be driven under its own power at about 2 miles per hour ; the normal speed when burning is 40 to 60 ft per hour. There is a mechanical digger incorporated in the front part of the machine to break up the previously consolidated formation into clods which are more easily burnt than a flat surface. It is sometimes found necessary to supplement this operation by using a tractor-drawn plough.

To construct a finished pavement, 12 ft wide and of reasonable depth of consolidation, the machine makes four parallel longitudinal runs, each 6 ft wide ; this produces a 24 ft wide strip burnt about 3½ in. deep loose measure. The outer 6 ft strips are turned in with a grader, thus giving one strip 12 ft wide. About 10 per cent of binder is generally required, this is added and the whole consolidated with a road roller. The resulting pavement is about 6 in. thick. Under certain conditions of soil and traffic it may prove necessary to increase the pavement thickness by burning additional areas and grading the material so produced on to the carriageway.

The hardness of the product of the heat treatment depends on the degree of compacting which has been done prior to burning. The treated material consolidates well, but is liable to abrasion under heavy traffic. It has been found advisable to seal the surface with bitumen to minimise wear. The cost per mile is about the same as that for gravelled roadway of equal dimensions under Australian conditions.

The cost per square yard of 6 in. finished pavement (double burnt) is about equal to 1½ hours wages for a labourer.

Although the Irvince road machine used in Australia indicates the possibility of the use of mechanical means of soil stabilization, in situ, by the heat treatment process, the machine in its present stage would need considerable improvement for greater speed of construction. Further details of this method are given in the Appendix.

ACKNOWLEDGMENT

The laboratory experiments reported in this Paper were carried out by Shri R. M. Palit, M. Sc., Senior Research Assistant under the direction of Shri A. K. Deb, M. Sc., Research Officer, Engineer Research Wing of the College of Military Engineers, Kirkee. The Author gratefully acknowledges the help rendered both by Shri Deb and Shri Palit in presenting the results of the investigation for this Paper.

Appendix

METHOD OF HEAT TREATMENT OF EARTH ROADS IN QUEENSLAND AND NEW SOUTH WALES, AUSTRALIA.

The machine, which consists essentially of a chassis on wheels carrying a furnace of special design, applies the heat directly to the soil on the road. The process is continuous and the machine moves over the road leaving behind as it moves a layer of the finished road surface.

The Heat Treatment process has been used with success for five years in Australia and the roads made using the process have proved capable of carrying fast and heavy traffic under all weather conditions. In many cases soil, which, in its natural state, becomes so boggy as to be impassable to traffic in wet weather economically treated.

The hardening effect is due to the fusion of certain constituents of the soil. The temperature generated by the machine is such that the soil can be completely melted. Complete fusion is not, however, advantageous and the movement of the machine is adjusted so that only partial fusion takes place. The soil is treated in its natural state and no extraneous material is added.

The bearing capacity of the subgrade is also somewhat increased by the stabilizing effect of a fraction of the heat which penetrates to the soil in the road-bed. There cannot be any well-defined separation between what may be regarded as road paving and the base or roadbed because the former gradually merges into the latter due to the gradual diminution of the temperature from the surface downwards.

The machine works continuously throughout the twenty-four hours and the crew consists of three operators working one at a time in shifts under a foreman. The weekly cost of operation under Australian conditions and currency is as follows:

Wages — 3 operators and 1 foreman	...	£ 25
Firewood-25 cords at £ 1, or equivalent in coal (1 ton of coal is approximately equal to 1 cord of wood in calorific value)	...	25
Plant charge, to include capital depreciation, interest and repairs	...	25
Fuel Oil for power unit	...	3
Lubricants	...	2
Sundries, say	...	5
Total for one week		£ 85

(At current exchange rate of 25 per cent = £ 68 sterling)

By substituting the equivalent values in wages, materials etc., the cost of working in any other country may be estimated. In estimating the number of working hours per week due allowance must be made for delays due to wet weather, running repairs and other causes. Under ordinary conditions 120 hours of work per week can be put in per week, based on a rate of movement of 40 feet per hour, the output will be 3,200 square yards per week. The rate of movement will vary between 30 and 80 feet per hour, acccording to the soil conditions and the effect required. The machine can be adjusted to move at any desired rate and the best rate should be determined by experience. The unit cost actually realised is, on latest figures, 7.35 d (5.88 d. sterling) per square yard for a single course of burning.

After the passage of the machine the treated material is left loose on the surface of the road. It is then consolidated by the addition of a small quantity of untreated soil, preferably of a loamy nature, which is mixed with the burnt material by means of an ordinary farm harrow with vertical prongs. The mixing requires to be done very thoroughly by repeated traverses of the harrow and consolidation is then completed with a light roller, or in the absence of a roller, under wheeled traffic. Watering is an advantage. The quantity of untreated soil to be added as a binder is roughly 200 cubic yards per mile of 12 foot road for a single course of burning.

Although the primary objective of Heat Treatment is to provide a low cost road in areas where the cost of bituminous work is not warranted by the conditions, it is considered that heat treated soil would form a good base for ordinary bituminous or tar seal work. In one instance, on a length of two miles of road, it has proved entirely satisfactory as a base for light bituminous penetration.

The combustion is carried out in two stages on the semi-producer gas principle. The primary stage consists of the reduction of the wood fuel to gas and charcoal, and the secondary stage of the burning of these in direct contact with the road, the surface of which has already been opened by an automatic device attached to the front of the machine so that the burning gases can enter the ground under pressure. The air for secondary combustion is preheated by contact with the hot product as it emerges from under the furnace and a large proportion of the heat which has already been used to heat the product is thus collected and used

again in the furnace. Besides saving heat this arrangement has the effect of "stepping up" the temperature of the secondary combustion.

(The information given above is extracted from "The Irvine Road Machine for making all weather roads by heat treatment of soil" and information supplied by the Secretary, Queensland Main Road Commission, Brisbane reference his letter of 19th September 1951 to the Engineer Research' Wing, Kirkee.).

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DIMENSIONS AND WEIGHTS OF ROAD-DESIGN VEHICLES

1. INTRODUCTION

1.1. The object of framing this Standard is to lay down a basis for designing road components. The dimensions and weights of vehicles are cardinal factors in the design of road elements. The width of the design vehicle has a bearing on the width of traffic lanes and that of shoulders. The height of the vehicle affects the clearance to be provided in designing road under-bridges, electrical service lines, and other overhead structures. The overall length of the vehicle (including trailer and semi-trailer combinations) has to be taken into consideration in designing horizontal curves, and vertical curves, as also in framing safety regulations for passing and overtaking. The axle load affects the design of the thickness of pavement, whereas the total weight of the vehicle governs limiting gradients.

1.2. At the Thirteenth Session of the Indian Roads Congress, Paper No. 138 on "Policy Concerning Maximum Dimensions, Weights, and Speeds of Motor Vehicles in India", presented by the Specifications and Standards Committee was discussed. The recommendations of the Committee were accepted and are now embodied in this Standard.

2. SCOPE

2.1. The Standard shall be applied in designing all road structures except culverts and bridges, the latter being governed by the I. R. C. Bridge Code.

2.2. The Standard shall not be taken to confer authority for over-riding any provisions of the Motor Vehicles Act of 1939 (as subsequently amended) or any rules framed thereunder.

2.3. Such of the maximum dimensions and weights, uot of those specified here, shall be used that have the severest effect in tht design of any road component. All road components, to be newly built or improved, shall be so designed that they are either initially adequate or are capable of being made adequate subsequently, when the necessity arises, for the movement of the vehicles conforming to this Standard.

3. DEFINITIONS

3.1. Axle load is the total load transmitted to the road by all wheels whose centres may be included between two parallel

vertical planes 40 in. apart, and transverse to the longitudinal axis of the vehicle.

3.2. Height, overall, is the perpendicular distance between the road surface and a plane parallel to the road surface through the topmost point of the vehicle. (In applying this, fire escapes, tower wagons, and other special features permitted by the competent authority shall not be taken into account.)

3.3. Length, overall, is the distance between two parallel planes normal to the road surface and passing through the extreme projecting points in front and at the rear of the vehicle. (This excludes the starting handle, any part of the hood when down, any ladder forming part of a turntable, fire escape, etc., fixed to the vehicle, and any post office letter box not exceeding 12 in. in length)

3.4. Width, overall, is the distance at right angles to the axis of the vehicle between two vertical parallel planes passing through extreme projecting points at the sides of the vehicle.

3.5. Trailer means any vehicle (other than a side-car) drawn or intended to be drawn by a motor vehicle (See Fig. 3). A semi-trailer is a trailer, some part of which rests on the motor vehicle that draws it (See Fig. 2).

3.6. Tractor means a motor vehicle (excluding a road roller which is constructed to draw a load.

4. DIMENSIONS OF ROAD-DESIGN VEHICLES

4.1. The maximum dimension are given below and are also illustrated in Figures 1 to 3:

	Maximum Dimensions.
Width	96 in.
Height— Single-decked vehicle	12 ft 6 in.
Double-decked vehicle	15 ft 6 in.
Length :	
(a) Single unit with two axles	35 ft 0 in.
(b) Single unit with more than two axles	40 ft 0 in.
(c) Semi-trailer-tractor Combination (Figure 2)	50 ft 0 in.
(d) Tractor and Trailer Combination (Figure 3).	60 ft 0 in.

No combination shall consist of more than two units, and no such combination, laden or unladen, shall have an overall length exceeding 60 feet.

5. WEIGHTS OF VEHICLES

5.1. No axle load shall exceed 18,000 lb.

5.2. The gross load of any vehicle or combination of vehicles shall not exceed that given by the following relation :

$$W = 1025\ (L + 24) - 3L^2$$

where W = the gross weight of the vehicle in lbs. ;

and L = the distance in feet between the extreme axles, measured parallel to the axis of the vehicle, to the nearest foot.

The above relation applies when L is not less than 8 ft. When L is less than 8 ft, the gross load shall not exceed 32,000 lb.

5.3. Maximum total loads of grouped axles shall conform to the following :

Distance between the extreme axles forming the group (in feet)	Maximum total total load of the group (in lb.)	Distance between the extreme axles forming the group (in feet)	Maximum total load of the group. (in lb)
4	32,000	32	54,328
6	32,000	34	55,982
8	32,608	36	57,612
10	34,550	38	59,218
12	36,468	40	60,800
14	38,362	42	62,358
16	40,232	44	63,892
18	42,078	46	65,402
20	43,900	48	66,888
22	45,698	50	68,350
24	47,472	52	69,788
26	49,222	54	71,202
28	50,948	56	72,592
30	52,650	57	73,278

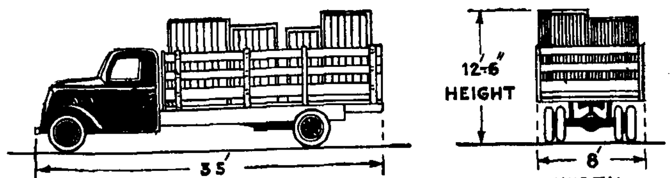
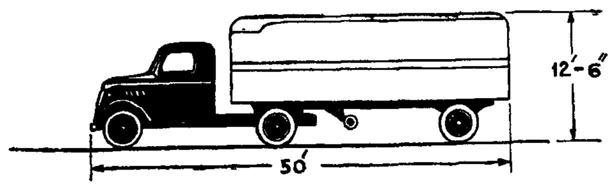
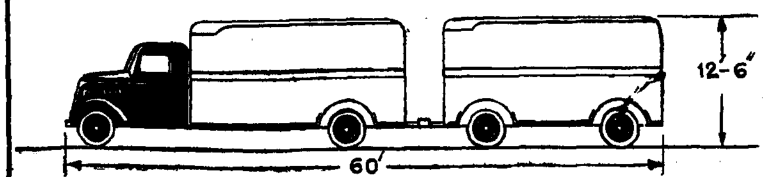


FIG. 1



TRACTOR SEMI-TRAILER COMBINATION

FIG. 2



TRACTOR AND TRAILER COMBINATION

FIG. 3.

Height of double decked Vehicle 15 ft 6 in.

ROUTE MARKER-SIGNS FOR NATIONAL HIGHWAYS

It has been considered desirable for various reasons that route-marking signs should be fixed along National Highways. Some type designs were worked out in the Roads Organization of the Ministry of Transport, Govt. of India, and discussed at the Chief Engineers' meeting held in April 1952, Fresh type designs, modified in the light of the Chief Engineers' suggestions, have been issued by the Consulting Engineer to the Govt. of India (Roads).

SPECIFICATIONS

1. A National Highway route-marker sign shall consist of a shield painted on a rectangular plate 18 in. by 24 in. The design is given in Plate I.

2. The sign shall have a **white or light yellow** background and the lettering shall be in black. The size and shape of the letters and numerals shall conform to those given in Plates II and III.

3. The sign shall be erected on National Highways at their junctions with other important roads, at suitable intervals through built-up areas, and at such other points that may be considered necessary to guide through traffic.

4. The sign shall be erected as indicated in the drawing titled "Arrangement for the erection of N. H. route-marker signs" (Plate IV).

5. The sign shall be erected at a distance not greater than 8 ft nor less than 3 ft from the edge of the carriageway.

6. When the sign is erected at a junction the direction which the National Highway takes at the junction shall be indicated on a **definition plate** of the size 12 in. by 10 in. The distance (along the National Highway) of the sign from the junction, on either side of it, shall be 350 to 500 ft. Also, it shall be erected on the left-hand side as one approaches the junction.

7. The sign may be of either enamelled or painted steel plate.

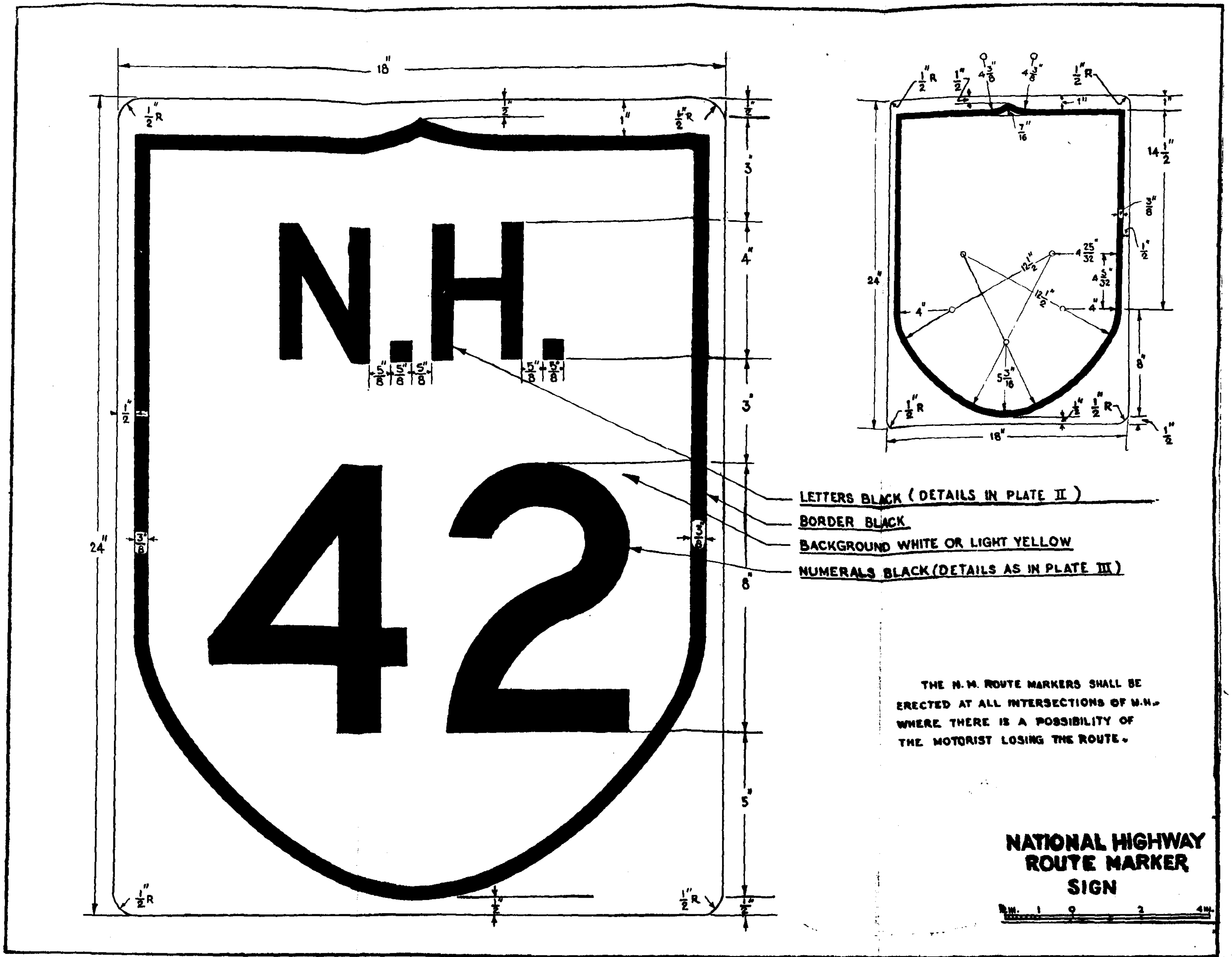
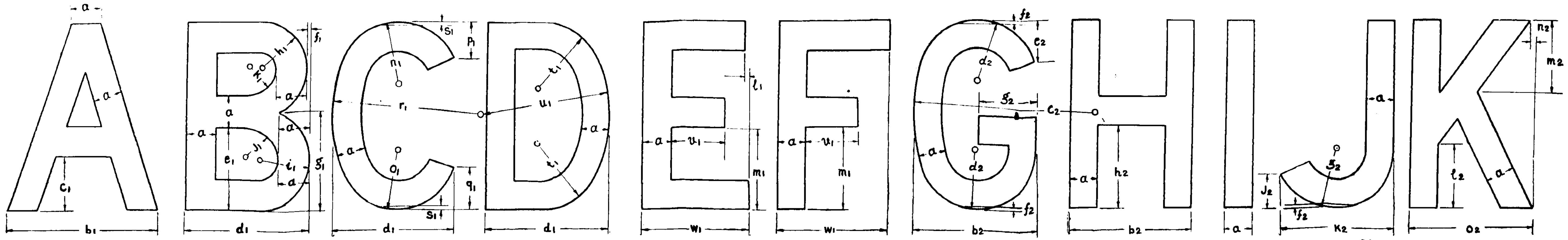
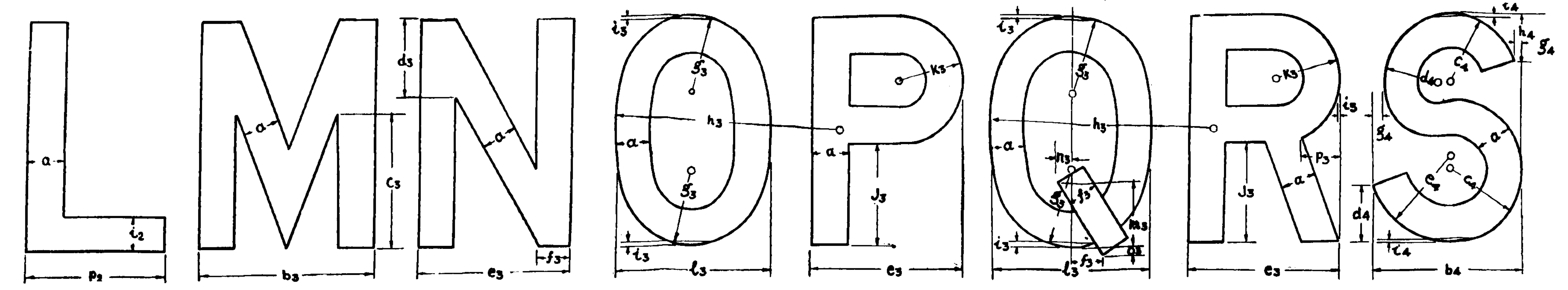
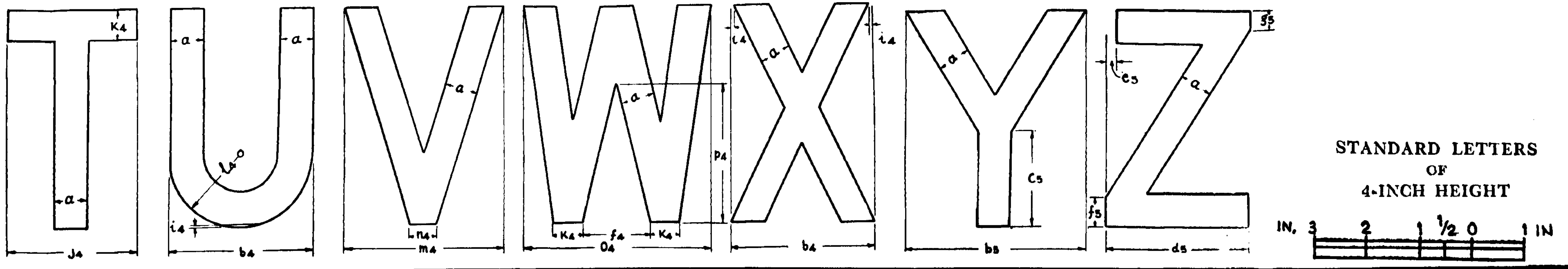

N.H.
42
LETTERS BLACK (DETAILS IN PLATE II)
BORDER BLACK
BACKGROUND WHITE OR LIGHT YELLOW
NUMERALS BLACK (DETAILS AS IN PLATE III)
THE N.H. ROUTE MARKERS SHALL BE ERECTED AT ALL INTERSECTIONS OF N.H. WHERE THERE IS A POSSIBILITY OF THE MOTORIST LOSING THE ROUTE.
NATIONAL HIGHWAY
ROUTE MARKER
SIGN

Plate II







KEY
SPACING FOR LETTERS

LETTER	LEFT	RIGHT	LETTER	LEFT	RIGHT
A	III	III	N	I	I
B	I	II	O	II	II
C	II	IV	P	I	II
D	I	II	Q	II	II
E	I	IV	R	I	I
F	I	IV	S	II	II
G	II	II	T	IV	IV
H	I	I	U	I	I
I	I	I	V	III	III
J	IV	I	W	III	III
K	I	IV	X	IV	IV
L	I	IV	Y	IV	IV
M	I	I	Z	IV	IV

RIGHT ⟶ LEFT	SPACING BETWEEN LETTERS
I ⟵ I I ⟵ II II ⟵ I	0 74
I, II ⟵ III, IV III, IV ⟵ I, II II ⟵ II	0.56
III, IV ⟵ III, IV OPPOSITE	0 38
III, IV ⟵ III, IV PARALLEL	0

SPACINGS BETWEEN LETTERS USE OF "KEY"

Example :—

DETERMINE THE SPACINGS BETWEEN 4-IN. LETTERS FORMING THE WORD DELHI

Solution :

D IS TO THE LEFT OF E UNDER THE COLUMN 'LEFT' IN THE KEY AGAINST D READ GROUP I. E IS TO THE 'RIGHT' OF D, AND AGAINST E, UNDER COLUMN 'RIGHT' READ GROUP IV. SO THE SPACE BETWEEN D AND E IS GIVEN AS 0.56 IN. AGAINST COLUMN

RIGHT⟷LEFT
I⟷IV

SIMILARLY SPACING BETWEEN E AND L WILL BE 0.56 IN. BETWEEN L AND H=0.74 IN. AND BETWEEN H AND I=0.74 IN.

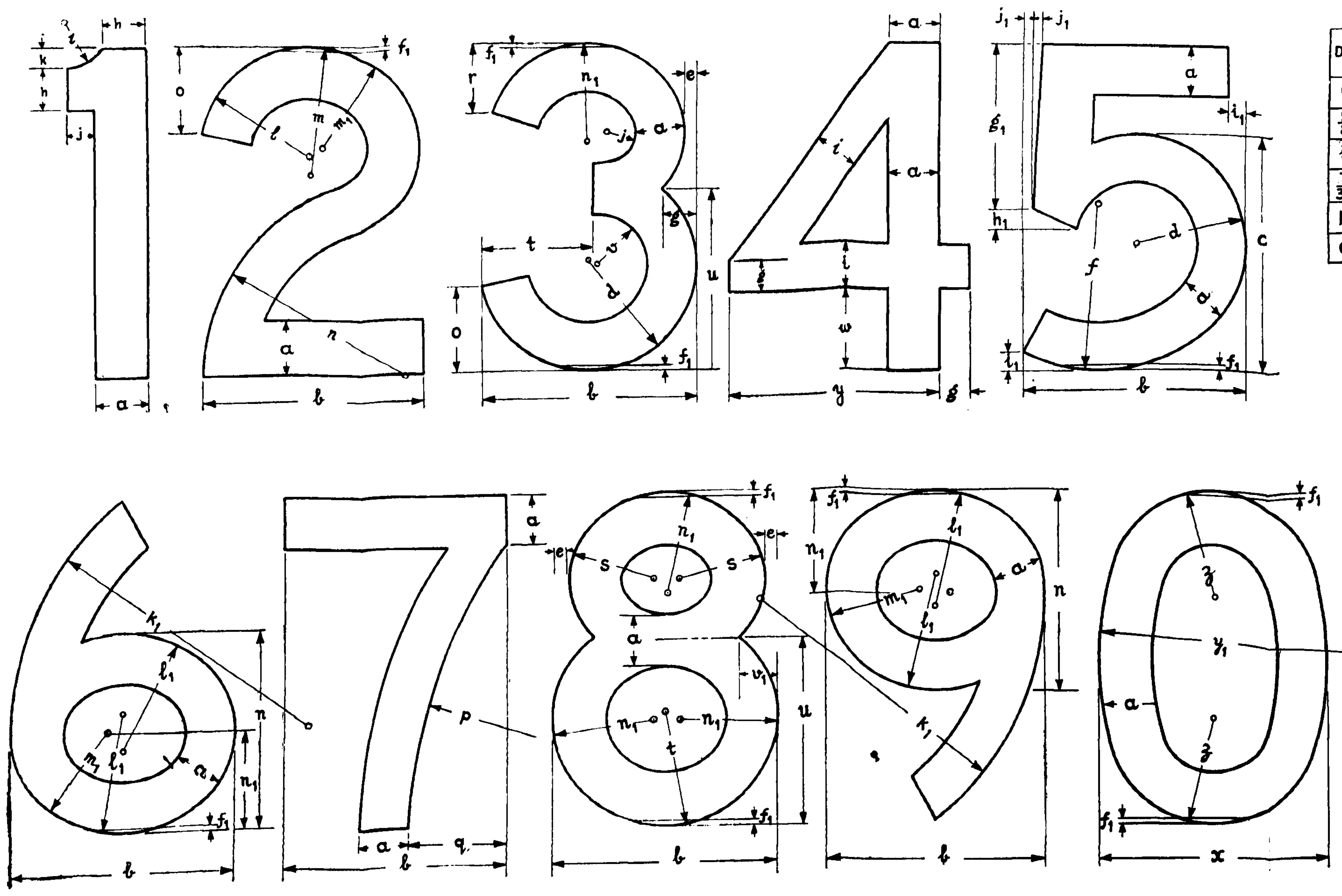
EXAMPLES

RIGHT ⟵ LEFT	LETTERS	NUMERALS
I ⟵ I I ⟵ II II ⟵ I	NI-UP-OH	11-22-21
I, II ⟵ III, IV III, IV ⟵ I, II II ⟵ II	IA-HT-OV-GX WR-KD-TB-YS SR-OQ	14-24-57-54 41-34-66-93
III, IV ⟵ III, IV OPPOSITE	WV-YV-EA-CT	96
III, IV ⟵ III, IV PARALLEL	WAY-LY-AT	69-67-76

DIMENSIONS (INCHES) FOR 4 INCH HEIGHT OF LETTERS

a	b_1	b_2	b_3	b_4	b_5	c_1	c_2	c_3	c_4	c_5	d_1	d_2	d_3	d_4	d_5	e_1	e_2
$\frac{5}{8}$	$3\frac{11}{32}$	$2\frac{11}{16}$	$3\frac{3}{32}$	$2\frac{11}{16}$	$3\frac{3}{8}$	$1\frac{1}{8}$	4	$2\frac{5}{16}$	$1\frac{7}{32}$	$1\frac{3}{4}$	$2\frac{11}{16}$	$1\frac{9}{32}$	$1\frac{11}{32}$	1	$2\frac{11}{16}$	$1\frac{3}{4}$	$\frac{7}{8}$
e_3	e_4	e_5	f_1	f_2	f_3	f_4	f_5	g_1	g_2	g_3	g_4	g_5	h_1	h_2	h_3	h_4	i_1
$2\frac{11}{16}$	$1\frac{9}{16}$	$\frac{5}{32}$	$\frac{3}{32}$	$\frac{1}{16}$	$\frac{9}{16}$	$1\frac{1}{4}$	$\frac{17}{32}$	$2\frac{3}{32}$	$1\frac{5}{16}$	$1\frac{9}{32}$	$\frac{5}{32}$	$\frac{3}{8}$	1	$1\frac{3}{4}$	4	$\frac{13}{16}$	$1\frac{5}{32}$
i_2	i_3	i_4	j_1	j_2	j_3	j_4	k_1	k_2	k_3	k_4	l_1	l_2	l_3	l_4	m_1	m_2	m_3
$\frac{15}{32}$	$\frac{1}{16}$	$\frac{1}{16}$	$\frac{21}{32}$	$\frac{23}{32}$	$1\frac{3}{4}$	$2\frac{7}{16}$	$\frac{5}{8}$	$2\frac{1}{2}$	$1\frac{1}{8}$	$\frac{9}{16}$	$\frac{1}{8}$	$1\frac{3}{8}$	$2\frac{13}{16}$	$1\frac{11}{32}$	$1\frac{3}{4}$	$1\frac{9}{16}$	$1\frac{1}{8}$
m_4	n_1	n_2	n_3	n_4	o_1	o_2	o_3	o_4	p_1	p_2	p_3	p_4	q_1	r_1	s_1	t_1	u_1
3	$1\frac{5}{16}$	$\frac{3}{32}$	$\frac{5}{16}$	$\frac{1}{2}$	$1\frac{9}{32}$	$2\frac{3}{4}$	$\frac{3}{16}$	$3\frac{1}{2}$	$\frac{13}{16}$	$2\frac{7}{16}$	$\frac{11}{16}$	$2\frac{9}{16}$	$\frac{7}{8}$	$3\frac{5}{32}$	$\frac{1}{16}$	$1\frac{7}{16}$	$2\frac{13}{16}$
									v_1	w_1							
									$1\frac{5}{32}$	$2\frac{7}{16}$							

Plate III



FOR HEIGHTS OF NUMERALS OTHER THAN 4 IN. THE DIMENSIONS GIVEN IN THE TABLE ARE TO BE PROPORTIONATELY INCREASED AND DECREASED.

DIMENSIONS (INCHES) FOR 4 INCH HEIGHT OF NUMERALS

a	b	c	d	e	f	f_1	g	g_1	h	h_1	i
$\frac{5}{8}$	$2\frac{11}{16}$	$2\frac{7}{8}$	$1\frac{11}{32}$	$\frac{5}{32}$	$2\frac{1}{32}$	$\frac{1}{16}$	$\frac{3}{8}$	$2\frac{1}{16}$	$\frac{1}{2}$	$\frac{3}{16}$	$\frac{9}{16}$
i_1	j	j_1	k	k_1	l	l_1	m	m_1	n	n_1	o
$\frac{7}{32}$	$\frac{11}{32}$	$\frac{3}{32}$	$\frac{1}{4}$	$3\frac{9}{16}$	$1\frac{5}{16}$	$1\frac{7}{16}$	$1\frac{9}{16}$	$1\frac{5}{32}$	$2\frac{7}{16}$	$1\frac{7}{32}$	$1\frac{1}{16}$
p	q	r	s	t	u	v	v_1	w	x	y	y_1
6	$1\frac{3}{16}$	$\frac{13}{16}$	$1\frac{1}{32}$	$1\frac{3}{8}$	$2\frac{1}{4}$	$\frac{19}{32}$	$\frac{15}{32}$	1	$2\frac{13}{16}$	$2\frac{9}{16}$	4
					z						
					$1\frac{9}{32}$						

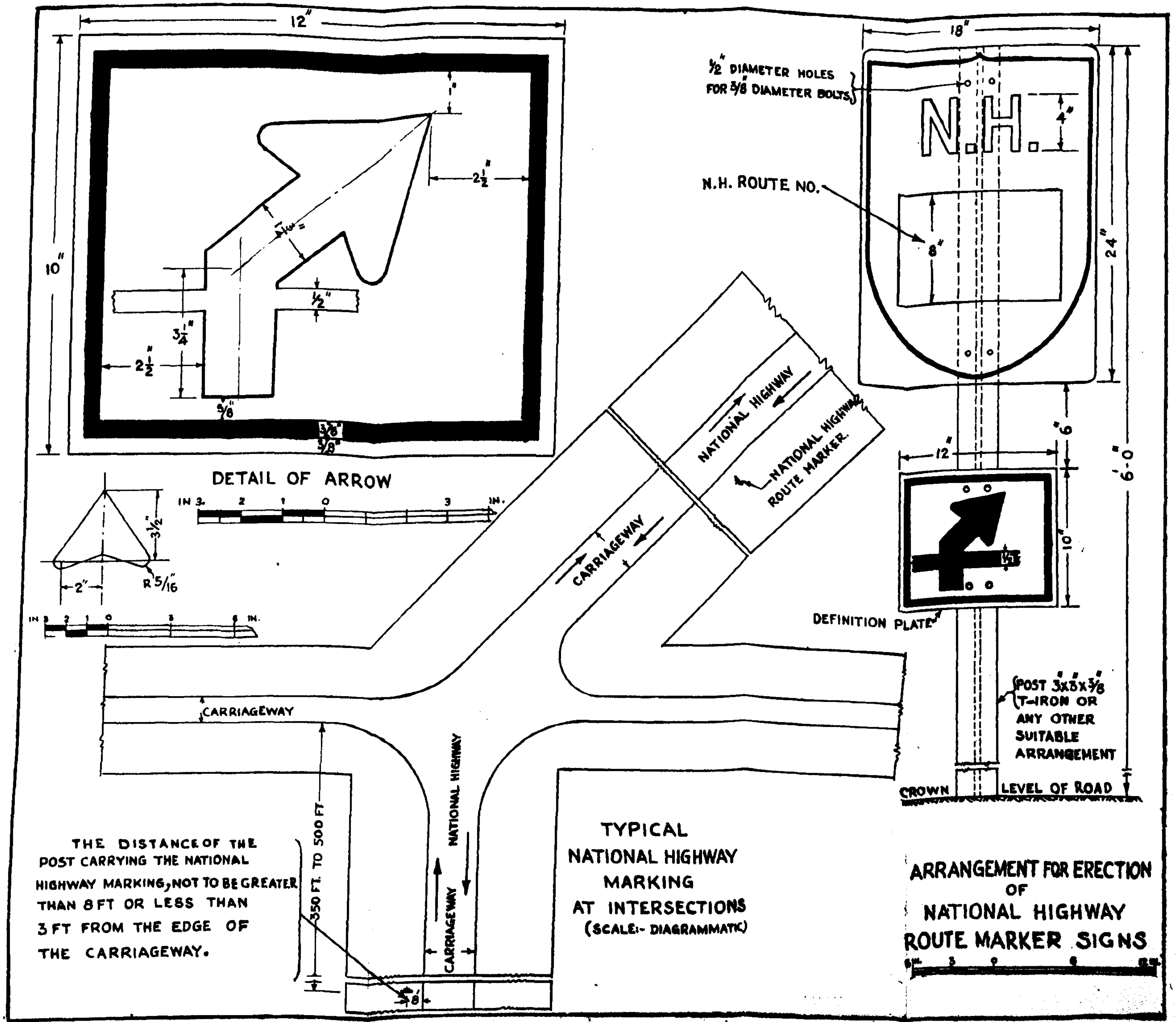
KEY
SPACING FOR NUMERALS

NUMBER	LEFT	RIGHT	RIGHT ↔ LEFT	SPACING BETWEEN NUMERALS
1	I	I	I ↔ I	0·74
2	II	II	I ↔ II	
3	IV	II	II ↔ I	
4	IV	IV	I II ↔ III IV	0·56
5	I	II	III IV ↔ I II	
6	II	II	II ↔ II	
7	IV	III	III IV ↔ III IV OPPOSITE	0·38
8	II	II		
9	II	II	III IV ↔ III IV PARALLEL	0·00
0	II	II		

STANDARD NUMERALS OF 4 INCH HEIGHT

SCALE:— 1 3/4 1/2 1/4 0 1 IN.

For obtaining dimension of 8 inch numerals multiply those given here by two.


12"
10"
1"
$2\frac{1}{2}$"
$\frac{1}{2}$"
$3\frac{1}{4}$"
$2\frac{1}{2}$"
$\frac{5}{8}$"
DETAIL OF ARROW
$3\frac{1}{2}$"
2"
R $\frac{5}{16}$"
18"
$\frac{1}{2}$" DIAMETER HOLES FOR $\frac{3}{8}$" DIAMETER BOLTS
N.H.
4"
N.H. ROUTE NO.
8"
24"
9"
6'-0"
12"
10"
DEFINITION PLATE
POST 3"x3"x$\frac{3}{8}$" T-IRON OR ANY OTHER SUITABLE ARRANGEMENT
CROWN
LEVEL OF ROAD
NATIONAL HIGHWAY
NATIONAL HIGHWAY ROUTE MARKER
CARRIAGEWAY
CARRIAGEWAY
NATIONAL HIGHWAY
CARRIAGEWAY
350 FT. TO 500 FT
8'
THE DISTANCE OF THE POST CARRYING THE NATIONAL HIGHWAY MARKING, NOT TO BE GREATER THAN 8 FT OR LESS THAN 3 FT FROM THE EDGE OF THE CARRIAGEWAY.
TYPICAL NATIONAL HIGHWAY MARKING AT INTERSECTIONS (SCALE:- DIAGRAMMATIC)
ARRANGEMENT FOR ERECTION OF NATIONAL HIGHWAY ROUTE MARKER SIGNS

A SUGGESTED CLASSIFICATION OF THE BLACK COTTON SOILS OF INDIA

Central Road Research Institute
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FOREWORD

The stabilisation of the Black Cotton Soils, or their conversion into a useful construction material, is not a simple problem in India. So far it has not been solved either satisfactorily or economically. The Indian Black Cotton Soils do not fit into the classification systems for soils given by the United States Public Roads Administration in 1942 and the United States Highway Research Board in 1945. May be this occurred because these soils were not taken into consideration, or thought to be uniform, or perhaps they were not looked upon as a possible construction material at all. It is realised that they are not uniform, but in large regions of India they are the only material available and have, therefore, to be used in road construction in one way or the other. Upto now no classification has been evolved to distinguish the different varieties of Black Cotton Soils which could help facilitate their handling.

Therefore it is necessary to know their properties so that it may be possible to employ these soils as useful materials of construction. With the object of evolving a classification system, an investigation on numerous soil samples collected from all those States of India where Black Cotton Soil is met with was undertaken. The result is a suggestion for an extension of the United States Public Roads Administration Classification System.

It is hoped that with the help of a more accurate method of classification it might be possible to treat these soils more successfully, in order to utilise them economically for the construction of roads and other construction purposes.

The studies were conducted as a part of the research programme of the Central Road Research Institute. The Director wishes to acknowledge the assistance given by Dr. H. L. Uppal and Mr. H. S. Bhatia in this study and the work done by Messrs K. L. Nayar, A. K. Bose, A. L. Chatwal and O. Mascaranhas in conducting the chemical and physical tests.

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Okhla, Delhi.

E. ZIPKES,
Director.

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A SUGGESTED CLASSIFICATION OF THE BLACK COTTON SOILS OF INDIA

1. INTRODUCTION

1.1. The Black Cotton Soils, which are heavy clay soils, are met with in tropical areas. In India they mostly occur in the central and southern parts of the country, particularly in the Deccan Plateau, where they are popularly known as *regur*.

1.2. The soils are characterised by their light to dark grey colour, their brittle behaviour when dry, and their stickiness and swelling when wet. When dry they have a high bearing capacity but develop wide shrinkage cracks on drying. They lose their bearing capacity almost completely when wet.

1.3. The ranges of the characteristics of these soils are given below :

Fine sand	3 to 10%
Passing 200 mesh	70 to 100%
Colloidal clay	40 to 50%
Liquid Limit	40 to 100%
Plasticity Index	20 to 60%
Volumetric shrinkage (on wet basis)	40 to 50%
Hygroscopic moisture	12% to 18%
Exchangeable Ca	40 to 50 m.e.*/100 gm soil.
Exchangeable Na+K	2 to 5 m.e./100 gm soil.
Base exchange capacity	40 to 50 m.e./100 gm soil.
pH	8 to 9%
$CaCO_3$	5 to 15%
SiO_3	50 to 55%
Fe_2O_3	8% to 12%
SiO_2/Al_2O_3	3% to 5%

Black Cotton Soils are generally shallow in depth and not different in chemical composition from their parent rock. There are large areas of transported soils which are often deeper than the surface crust and do not show much variation in profiles. These soils generally lie over *Kankar* beds.

* milli-equivalent.

1.4. Much work has been done on the Black Cotton Soils in the Indian Agricultural Research Institute. This work does not, in all cases, include properties of the soils for construction purposes, particularly for roads.

1.5. The object of this study is to find a more detailed and therefore more suitable classification in order that it may be possible to utilize the classification for construction purposes. The grouping of these soils into one or more groups will thus be helpful in the treatment of black cotton soils for stabilization purposes.

2. COLLECTION OF THE SOILS

In 1945 the Roads Wing of the Ministry of Transport, Government of India, arranged for the collection of small samples from the black cotton soil areas, of the States of Bombay, Hyderabad and Madhya Pradesh. These samples were inadequate in quantity as well as in variety and therefore a large number of samples were subsequently arranged for by the Central Road Research Institute from these States in the years 1950 to 1952. The various places from where the samples were collected are shown in Plate 1. In all 210 samples were investigated.

3. CHEMICAL ANALYSIS

3.1. Fifty two samples from Hyderabad State were subjected to chemical tests and the following determinations were made :—

1. Total soluble solids. Determination by the evaporation method with 1 : 5 soil-water suspension.
2. Carbonates. Carbonates which generally exist as calcium carbonate were determined by Schroter's apparatus.
3. Exchangeable calcium. The determinations were carried out with acetate-oxalate-carbonate mixture.[1]
4. Exchangeable Na+K. The exchangeable Na+K was determined by Ammonium Carbonate method.[2]
5. pH. pH was determined with Beckman pH-meter.
6. Organic carbon. The organic carbon was determined by Walkley and Black's rapid titration method.[3]

1. Refers to the Serial Nomber in References.

3.2. The results of the above-mentioned tests are given in Table 1.

The results do not show any tendency of the soils of separating themselves into different groups. It can be concluded that chemical analysis does not provide a sure means of dividing the soils in so far as their behaviour in the field is concerned. Therefore chemical analysis of the soils from the other two States, namely Bombay and Madhya Pradesh, was not carried out. Further studies were confined to the investigation of the physical and mechanical properties of the soils.

4. CLASSIFICATION OF SOILS ON THE BASIS OF THEIR PHYSICAL PROPERTIES

4.1. During the last 25 years a number of systems of classification of soils for engineering purposes have been developed.

Some of these are:

1. Textural classification; (based on grain size classification and distribution).
2. Casagrande's classification; (based on suitability for carrying load).
3. U. S. Public Roads Administration classification; (based on suitability for road surfacings and bases; using the liquid limit, plasticity index, mechanical analysis and group indexes).
4. Civil Aeronautic Administration classification; (based on mechanical analysis, plasticity, expansive qualities, California bearing ratio and general description of soil based on field examination).
5. Compaction classification; (based on maximum compaction attained by a soil).
6. Burmister classification; (based on grain size classification and distribution).

4.2. In 1952 the Working Committee of the Indian Roads Congress on soil classification approved of the U. S. Public Roads Administration (P.R.A.) classification for adoption for Indian soils on the consideration that the system had primarily been evolved for road purposes, and was comparatively simple. The system is briefly described in the following paragraphs.

5. U. S. PUBLIC ROADS ADMINISTRATION CLASSIFICATION

5.1. This system of classification was adopted by the U. S. Bureau of Public Roads in 1942.[4] In the system (1942 version) all the different soils were divided into eight groups, ranging from A-1, a well-graded gravel-sand-clay mixture, to A-8, a peat.

5.2. The system is based on the following six properties of the soil, the tests for which are carried out on material passing A.S.T.M. Sieve No. 40 :—

1. Particle size distribution ;
2. Liquid Limit ;
3. Plasticity Index ;
4. Shrinkage Limit ;
5. Field moisture equivalent; and
6. Centrifuge moisture equivalent.

5.3. This system of classification of 1942[4] was revised by the United States Highway Research Board in 1945[5]. The number of groups was reduced from eight to seven and, of the six tests listed in para 5.2, only the first three were used. In Table 2 the seven revised groups are indicated.[5] These seven groups were further converted into twelve sub-groups which are given in Table 3.

Figure 1 shows the ranges for the groups A-4, A-5 and A-7 (1945 version) as per Ref. 5, page 378. The limits of the group A-7 are added which are shown in the chart of soil characteristics for identification of fine grain size by the original U.S.P.R.A. System (revised version 1942) from Ref. 7, page 73.

5.4. The P. R. A. Classification, version 1945, (Ref. 5) limits group A-7 to Plasticity Index above 10 and Liquid Limit above 40. In the same Reference we find a description of the soils belonging to the group A-7 (similarly found in Ref. 9 on page 55) :

" The typical material of this group is similar to that described under group A-6, except that it has the high liquid limits characteristic of the A-5 group and may be elastic as well as subject to high volume change. The range of group index values is 1 to 20 with increasing values indicating the combined effect of in-

TABLE 2.

HIGHWAY RESEARCH BOARD CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES (REVISED VERSION 1942)

General Classification	Granular Materials (35 per cent or less passing No. 200)			Silt-Clay Materials (More than 35 per cent passing No. 200)			
Group Classification	A-1	A-3[a]	A-2	A-4	A-5	A-6	A-7
Sieve analysis—per cent passing :							
No. 10 sieve	—	—	—	—	—	—	—
No. 40 sieve	50 max.	51 min.	—	—	—	—	—
No. 200 sieve	25 max.	10 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40 sieve :							
Liquid Limit.		non plastic	—	40 max.	41 min.	40 max.	41 min.
Plasticity index.	6 max.		—	10 max.	10 max.	11 min.	11 min.
Group Index	0		4 max.	8 max.	12 max.	16 max.	20 max.
General rating as subgrade.	Excellent to good			Fair to poor			

[a] The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

TABLE 3.

HIGHWAY RESEARCH BOARD CLASSIFICATION OF SOILS AND SOIL-AGGREGATE MIXTURES. (WITH SUGGESTED SUBGROUPS) (VERSION 1945)

General Classification	Granular materials (35 per cent or less passing No. 200 sieve)							Silt-Clay Materials (More than 35 per cent passing No. 200 sieve)			
Group Classification	A–1		A-3	A-2				A–4	A–5	A–6	A–7
	A-1-a	A-1-b		A-2-4	A-2-5	A-2-6	A-2-7				A–7–5 A–7–6
Sieve analysis-Per cent pasing											
No. 10 sieve	50 max.										
No. 40 sieve	30 max.	50 max.	51 min.								
No. 200 sieve	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40 sieve :											
Liquid limit				40 max.	41 min.	40 max.	41 min.	40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		N. P.	10 max.	10 max.	11 min.	11 max.	10 max.	10 max.	11 min.	11 min[a]
Group Index.	0		0	0		4 max.		8 max.	12 max.	16 max.	20 max.
Usual types of significant constituent materials	Stone fragment gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General rating as subgrade	Excellent to good					Fair to poor					

[a] Plasticity index of A–7–5 subgroup is equal to or less than LL minus 30. Plasticity index of A–7–6 is greater than LL minus 30.

creasing liquid limits and plasticity indexes and decreasing percentages of coarse material ".

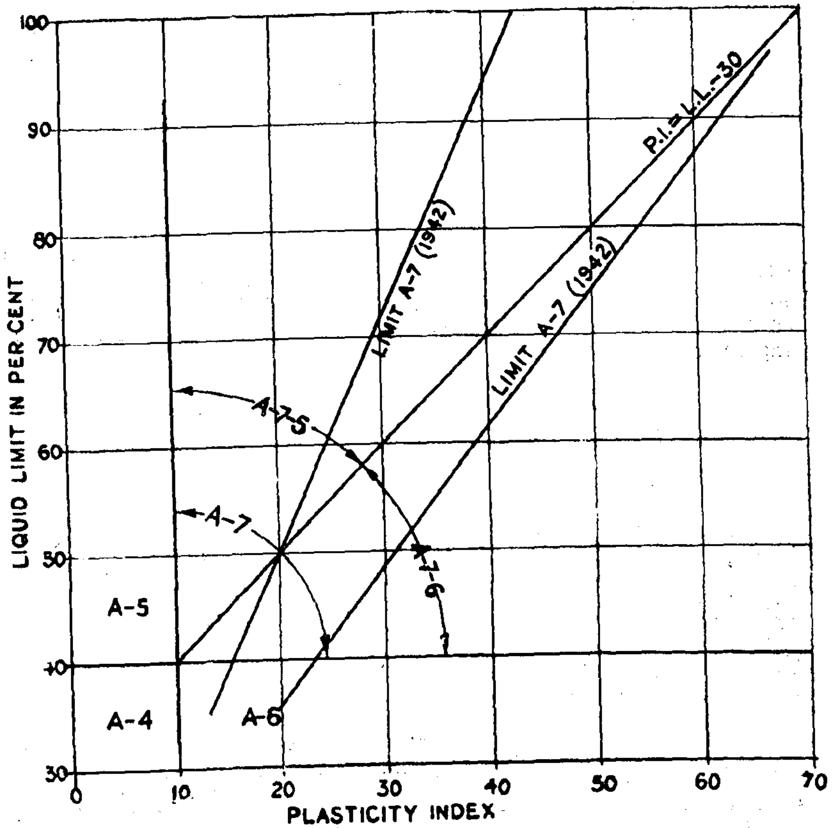


FIGURE 1. LIQUID LIMIT AND PLASTICITY INDEX RANGES FOR THE A-4, A-5, A-6 AND A-7 SUBGRADE GROUPS. AS PER PRA-SYSTEM 1945 AND LIMITS OF THE A-7 GROUP AS PER PRA-SYSTEM 1942 (REF. 5 PAGE 378 AND REF. 7 PAGE 73).

Group A–7 covers a large range; therefore a line for P.I. = L.L. – 30 was plotted and is shown in Figure 1. This line divides the group A–7 into two sub-groups, A–7–5 and A–7–6. These two sub-groups are defined as under :

" Sub-group A–7–5 includes those materials, with moderate Plasticity Index in relation to Liquid Limit and which may be highly elastic as well as subject to considerable volume change".

" Sub-group A-7-6 includes those materials with high Plasticity Index in relation to Liquid Limit and which are subject to extremely high volume change " (Ref. 5, page 378/9 and Ref. 9, page 56).

6. GROUPS INTO WHICH THE INDIAN BLACK COTTON SOILS SHOULD BE DIVIDED

6.1. The present study investigates some of the Black Cotton Soils of India, following the P.R.A. System (1945). All the samples from the States, Hyderabad (56 samples), Bombay (86 samples) and Madhya Pradesh (68 samples) were analysed according to the three tests :—

(1) Fraction passing A.S.T.M. Sieve No. 200.

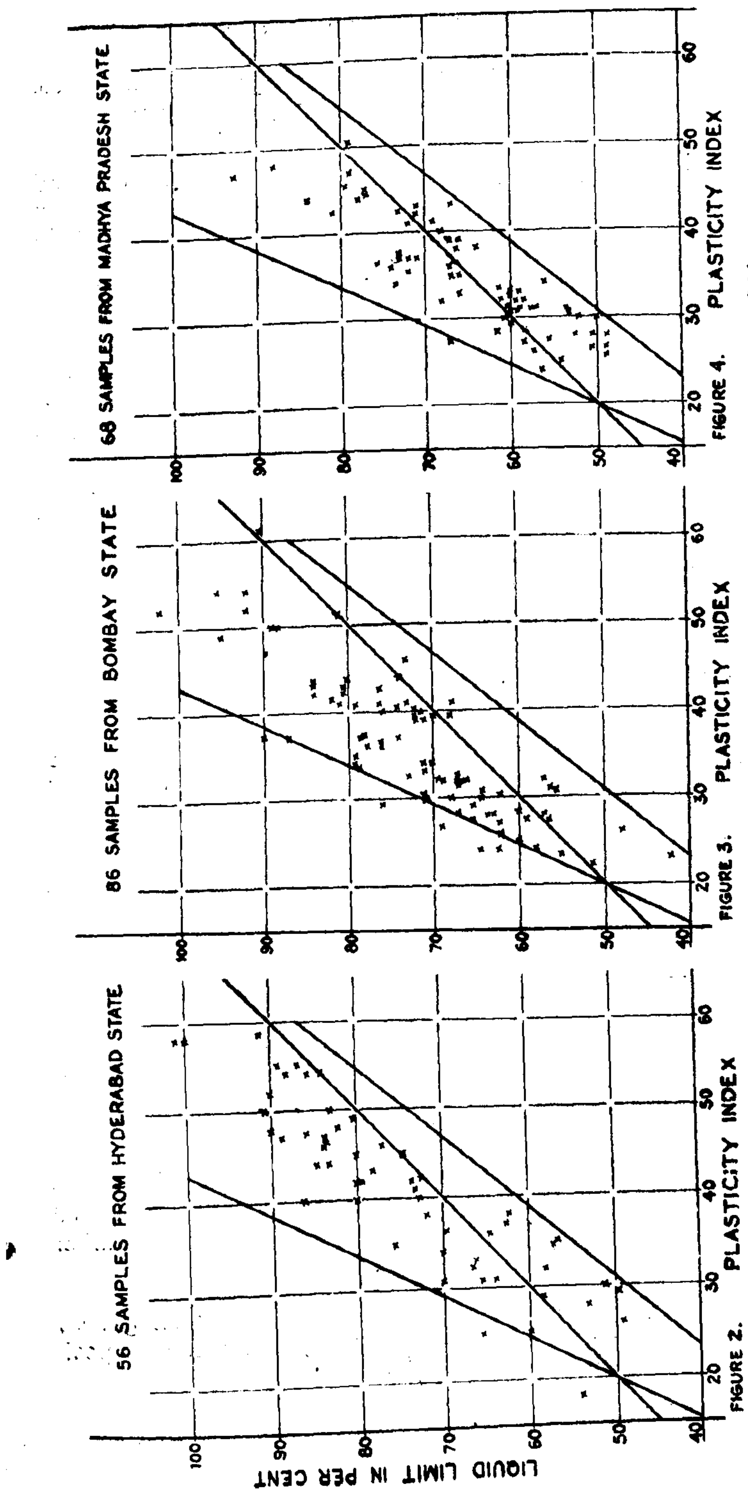
(2) Liquid Limit.

(3) Plasticity Index.

The results of the analysis are given separately for each State; for Hyderabad in Table 4, for Bombay in Table 5, and for Madhya Pradesh in Table 6. The results are also presented in Figures 2, 3 and 4 respectively for Hyderabad, Bombay and Madhya Pradesh. The values of liquid limit are plotted against the plasticity indexes within the limits given by the U.S.P.R.A. classification 1942 and 1945 according to Figure 1.

6.2. As can be seen from Figures 2, 3 and 4 all the 210 soils investigated fall into the A-7 group of the U.S.P.R.A. classification of 1945. Some of them are in sub-group A-7-5 and the rest in sub-group A-7-6, but all of them are relatively near to the line, P.I. = L.L. − 30. With certainty, however, we find them between the limits of the A-7 group defined according to the revised 1942 version with 1.27 × P.I. = L.L. − 11.86 and 2.14 × P.I. = L.L. − 7.5 which are used in Figures 2, 3 and 4 (as per Ref. 7, page 73), whereas these were defined earlier in 1940 by Plummer and Dore [8], as 1.6 × P.I. = L.L. − 14 and 1.02 × P.I. = L.L. − 14.

6.3. It is seen that, as far as these Black Cotton Soils are concerned, the limits of the A-7 group of the 1942 version are more precise than those of the 1945 version which are far too wide. The additional sub-grouping into groups A-7-5 and A-7-6 does not help in classifying these soils. They give no improvement



RELATION BETWEEN LIQUID LIMIT AND PLASTICITY INDEX against the A-7 limits of the revised version 1942 because both the sub-groups allow too large a variation in the liquid limit.

6.4. The results of this investigation show that the soil samples vary in their physical properties considerably even though they are in the same group A-7; for instance their liquid limit varies from 40 to 100, their plasticity index from 20 to 60, and the fraction passing A. S. T. M. Sieve No. 200 from 70 to almost 100 per cent.

6.5. Since the composition of the soils is so different their physical behaviour is bound to vary accordingly. It is, therefore, too big a generalisation to consider them all as belonging to one group A-7 or even to the two sub-groups, A-7-5 and A-7-6. Moreover there seems to be no indication that the line P.I. = L.L. - 30 as mentioned above does divide the soils into two significant groups. For an economical treatment of Black Cotton Soils, it would definitely be of advantage to be able to subdivide the main group A-7 into more distinct sub-groups.

7. GROUP INDEX AS A CLASSIFYING SYSTEM

7.1. In the course of the "Symposium on the identification and classification of soils", Atlantic City 1950, E. A. Willis[9] explained the group index as follows:

"The empirical group index formula devised for approximate within-group evaluation of the clayey-granular materials and the silt-clay materials is expressed as follows:

Group index $= 0.2a + 0.005\,ac + 0.01\,bd$

where

a = that portion of percentage passing No. 200 sieve greater than 35 and not exceeding 75, expressed as a positive whole number (1 to 40);

b = that portion of percentage passing No. 200 sieve greater than 15 per cent and not exceeding 55 per cent, expressed as a positive whole number (1 to 40);

c = that portion of the numerical liquid limit greater than 40 and not exceeding 60, expressed as a positive whole number (1 to 20); and

d = that portion of the numerical plasticity index greater than 10 and not exceeding 30, expressed as a positive whole number (1 to 20).

"The group index is based on the following assumption:

(a) Materials falling within Groups A-1-a, A-1-b, A-2-4, A-2-5 and A-3 are satisfactory as sub-grade when properly drained and compacted under moderate thickness of pavement (base and/or surface course) of a type suitable for the traffic to be carried, or can be made satisfactory by additions of small amounts of natural or artificial binders.

(b) Materials falling within the "clayey granular" Groups A-2-6 and A-2-7 and the "silt-clay" Groups A-4, A-5, A-6 and A-7 will range in quality as sub-grade from the approximate equivalent of the good A-2-4 and A-2-5 sub-grades to fair and poor sub-greades requiring a layer of sub-base material or an increased thickness of base course over that required under (a) in order to furnish adequate support for traffic loads.

(c) The assumed critical ranges of percentages passing the No. 200 are 35 to 75 neglecting plasticity and 15 to 55 as affected by plasticity indexes greater than 10.

(d) The assumed critical range of liquid limit is 40 to 60.

(e) The assumed critical range of Plasticity index is 10 to 30.

"The formula will give values ranging from a fraction of 1 to 20, and is so weighted that the maximum influence of each of the three variables is in the ratio of 8 for per cent passing the No. 200 sieve, 4 for liquid limit, and 8 for plasticity index. This weighting and the adopted critical ranges represent the best judgement of the committee based on the study of average relative evaluation placed on subgrade materials by several highway organizations which use the tests involved in this classification system.

"Under average conditions of good drainage and thorough compaction the supporting value of a material as subgrade may be assumed as an inverse ratio to its group index, that is, a group index of 0 indicates a "good" subgrade material and group index of 20 indicates a "very poor" subgrade material".

7.2. The group index is also determined from the dotted portions of the two charts given in Fig. 5 by taking the sum of the readings of both the charts. (The dotted lines give the original charts for the calculation of the group indexes 1-20).

7.3. It may be seen from Table 3 (classification version 1945) that soils of group A-6 are allowed to have a group index upto a maximum of 16, and soils of group A-7, including both the sub-groups A-7-5 and A-7-6, upto a maximum of 20. It therefore appears that soils with the group index from 16 to 20 are shown in group A-7.

7.4. Having applied the method of group indexes to the Black Cotton Soils from the three States, it was found that all the soils, with only a few exceptions, obtain a group index of more than 20, which is the maximum limit for the group A-7. Therefore the method of group indexes, at its present stage, is not helpful for a more precise classification of the Black Cotton Soils investigated. There seems to be a slight contradiction that these soils belong definitely to the A-7 group covering a wide area in between the limits for the A-7 group, and yet they do not find an exact place in the system of group indexes, which should be of additional and supplementary character.

8. EXTENSION OF THE GROUP INDEXES

8.1. As the original group index charts (vide Fig. 5) are calculated for group indexes upto a maximum of 20, an extension of these charts was designed by fixing higher limiting values for the fraction passing through A. S. T. M. Sieve No. 200, liquid limit, and plasticity index. This was done by raising the critical ranges of percentages passing sieve No. 200 from 35-75 to 35-100 and from 15-55 to 15-80, the liquid limit range from 40 to 60 to 40-85, and the plasticity index range from 10-30 to 10-44. The factors for the calculation of the group indexes, following the original equation as given in para 7.1, are now:—

a = that portion of percentage passing No. 200 sieve greater than 35 and not exceeding 100, expressed as a positive whole number (1 to 65);

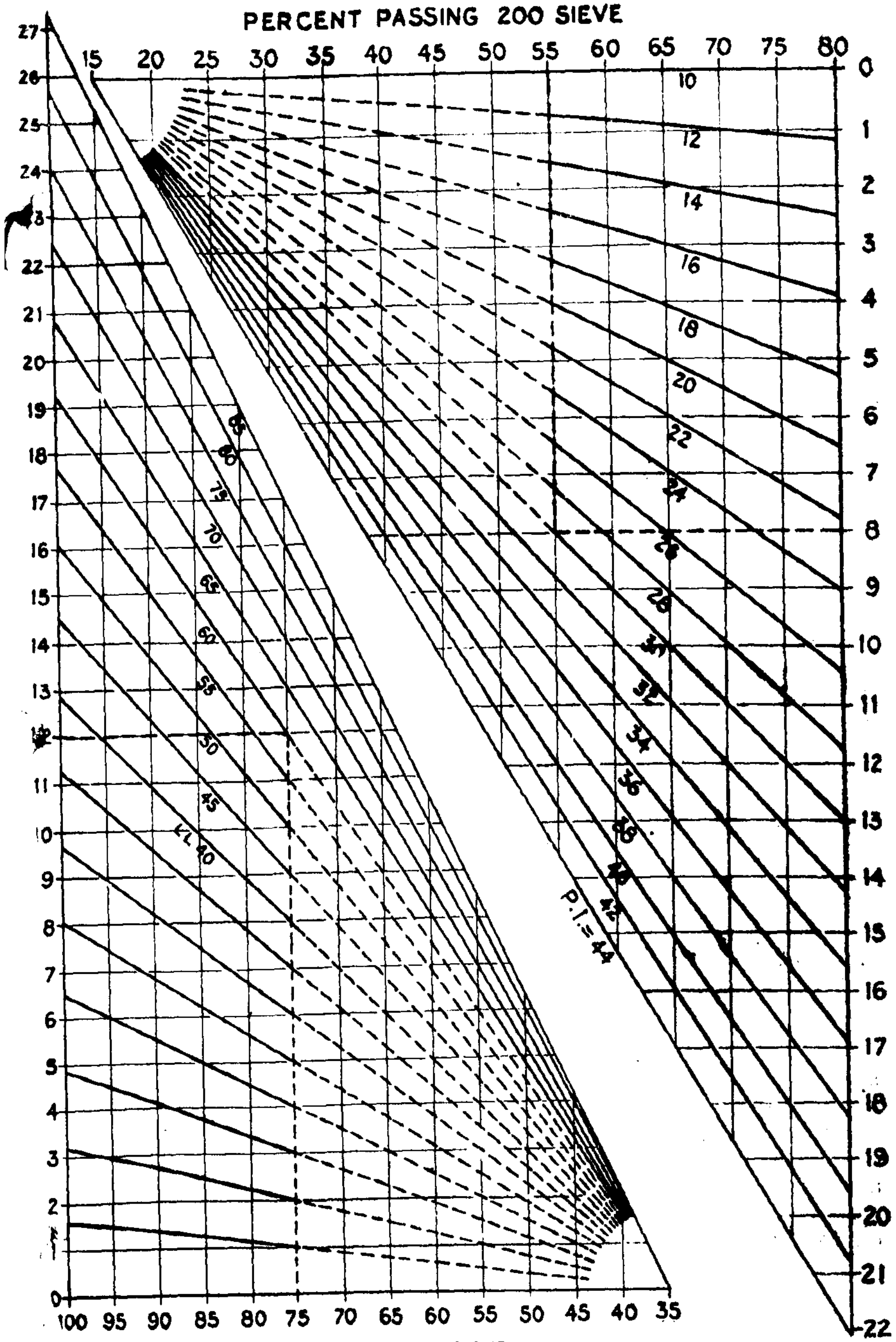


FIGURE 5. GROUP INDEX CHARTS
DOTTED LINES: CHARTS FOR THE GROUP INDICES 1-20

b = that portion of percentage passing No. 200 sieve greater than 15 per cent and not exceeding 80 per cent, expressed as a positive whole number (1 to 65);

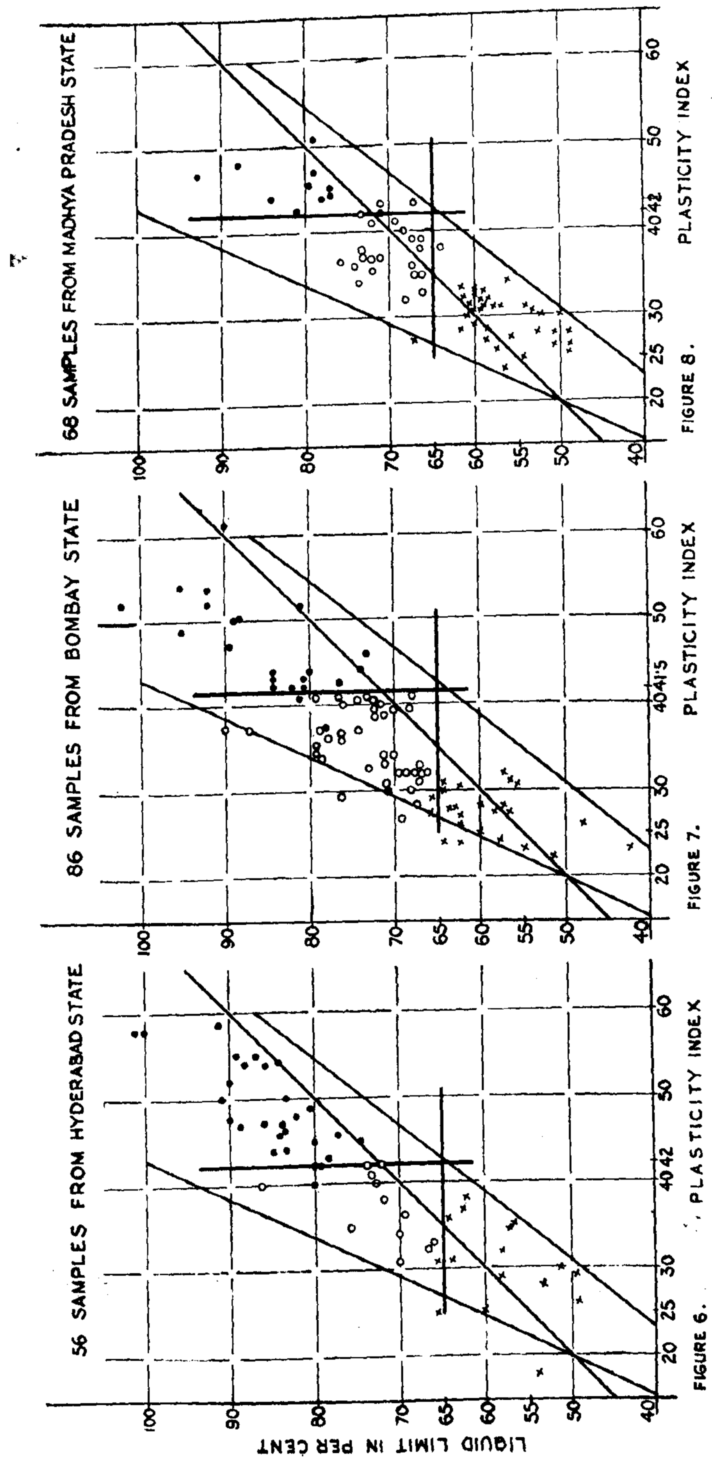
c = that portion of the numerical liquid limit greater than 40 and not exceeding 85, expressed as a positive whole number (1 to 45); and

d = that portion of the numerical plasticity index greater than 10 and not exceeding 44, expressed as a positive whole number (1 to 34).

8.2. In order to limit the group indexes to a maximum of 50 the Liquid Limit as well as the Plasticity Index have been fixed as shown. Values exceeding the above limits were considered as having the highest value fixed. The two charts given in Fig. 5 have been extended, keeping proportionally the same scales.

8.3. With these revised charts the group indexes of the Black Cotton Soils investigated were determined and are shown in Table 7, 8 and 9, arranged according to the group indexes 20 to 30, 30 to 40, and 40 to 50 respectively. The forms used in Figures 2, 3, and 4 were adopted to plot these results in Figures 6, 7 and 8 respectively for the three States with the distinctive symbols for each sub-group: the sub-group with group indexes 20-30 with crosses, those of 30-40 with hollow circles, and those of 40-50 with dots.

8.4. Figures 6, 7 and 8 show a distinct separation of the soils following the three sub-groups according to their group indexes. First a horizontal line drawn identically in all these figures on the liquid limit 65 divides one sub-group from the other two. An almost clear partition of the remaining sub-groups can be found horizontally in an area between the plasticity index values of 40.5-43, which might be taken as a boundary area. The dividing line itself can be located with some exactness at the plasticity index 42. As the exact amount of average error was not determined the plasticity index of 42 was assumed as a provisionally fixed value. It may be shifted later on to the calculated place within the limits mentioned above.



RELATION BETWEEN LIQUID LIMIT, PLASTICITY INDEX AND THE GROUP INDICES 20-30 (×); 30-40 (o) AND 40-50 (•) OF THE INVESTIGATED SOILS.

8.5. Figure 9 shows, therefore, the space which accommodates all samples investigated with relatively moderate liquid limit (about 55 to 65) and plasticity indexes (about 25 to about 42) with the group indexes 20–30. The remaining samples with a high liquid limit are further divided into those with relatively low (about 25 to about 42.0) and high (above 42.0) plasticity indexes; the lower part having a group index from 30 to 40 and the higher part with group indexes from 40 to 50.

8.6. The soils of the previous group A-7 (1945 version), having group indexes upto a maximum of 20, can also be easily

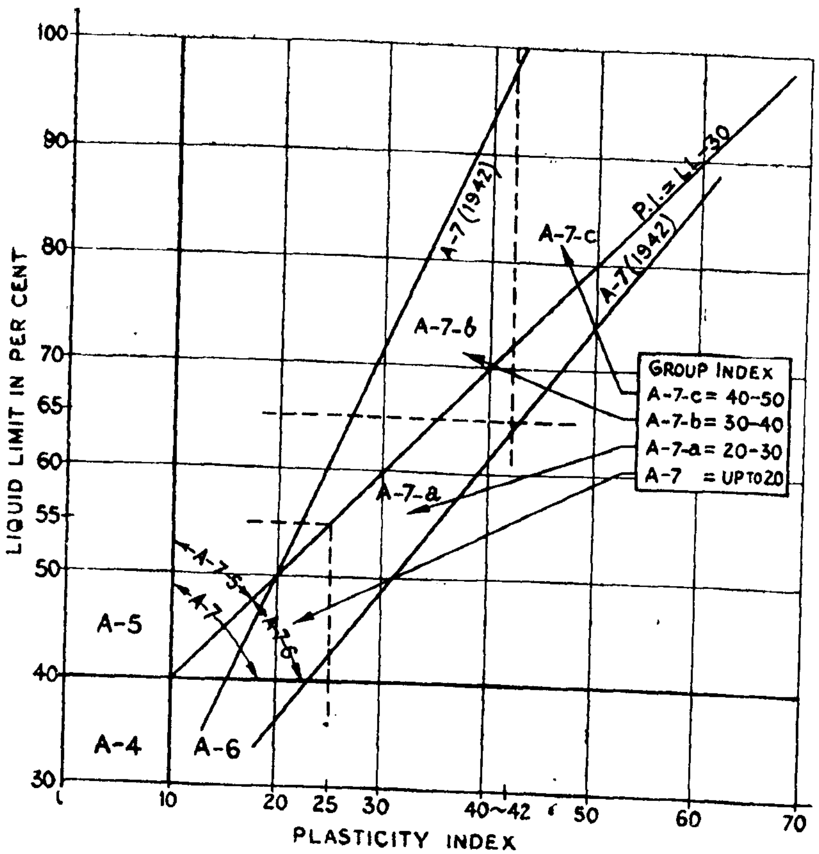


FIGURE 9. ANALOGICALLY TO FIGURE 1. SHOWING THE GROUP RANGES FOR THE A-4, A-5, A-6, A-7 AS WELL AS THE SUGGESTED THREE SUB GROUPS A-7-a, A-7-b, AND A-7-c ACCORDING TO THEIR LIQUID LIMITS, PLASTITY INDEX AND THEIR GROUP INDICES.

placed in the space given by the liquid limit lower than about 55 and the plasticity index lower than about 25. Table 10 shows in a comparative statement the groups A-6 and A-7 (1945 version) next to the suggested sub-groups.

8.7. Therefore the soil may be characterised by group indexes as under:

	Soils with a group index	max 20 named A-7
	Soils with a group index	20–30 named A-7a
	Soils with a group index	30–40 named A-7b
and	Soils with a group index	40–50 named A-7c

The soils are also given with the symbols of their sub-groups in Plate 1 which indicates the localities where the samples were taken.

8.8. Described in the terms of their physical properties the three sub-groups will contain soils as follows:

A-7a will accommodate soils with liquid limit of 55 to 65 and a P. I. of 25 upto about 42. Its fraction passing the ASTM sieve No. 200 ranges from 55 to 95 per cent (as per Tables 2, 3 and 4).

A-7b will accommodate soils with liquid limit higher than 65 and a P. I. upto about 42. Its fraction passing the ASTM sieve No. 200 ranges from 80 to 95 per cent (as per Tables 2, 3 and 4).

A-7c will accommodate soils with liquid limit above 65 and a P. I. above about 42. Its fraction passing the ASTM sieve No. 200 ranging from 85 to 100 per cent (as per Tables 2, 3 and 4).

9. COMPARISON OF THE SUB-GROUPS WITH THE RESULTS OF OTHER PHYSICAL TESTS

9.1. An attempt was made to compare the proposed sub-division of Black Cotton Soils with the results of some other physical tests, because the method with which the group indexes were calculated-although based on physical soil properties–follows an empirical formula. Therefore most of the soil samples were subjected to the shrinkage limit, the field moisture equivalent, and the hygroscopic moisture absorption tests. These tests, with the exception of the third, are accepted and recommended by the American Association of the State Highway Officials owing to the

TABLE 10

COMPARISON OF GROUPS A-6 AND A-7 (VERSION 1945) WITH THE SUGGESTED SUB-GROUPS A-7, A-7a, A-7b and A-7c.

System	Highway Research Board Classification of Soils (with suggested subgroups) 1945 version			Proposed clasification of Indian Black Cotton Soils			
		A-7					
Group	A-6	A-7-5	A-7-6	A-7	A-7a	A-7b	A-7c
Per cent passing 200 sieve	36 min.	36 min.		Upto 55	55–95	80–95	85–100
Characteristics of fraction passing No. 40 sieve. Liquid Limit. Plasticity index.	40 max, 11 min.	41 min. 11 min[a]		below 55 below 25	below 65 below 42	above 65 below 42	above 65 above 42
Group Index.	16 max.	20 max.		max. 20	20–30	30–40	40–50
Usual type of significant constituent materials.	Clayey soils.	Clayey soils.		Clayey soils			
General rating as subgrade.	fair to poor.	fair to poor.		fair to poor.	poor.		

[a] Plasticity index of A-7-5 subgroup is equal to or less than I.L minus 30. Plasticity index of A-7-6 is greater than L.L minus 30.

fact "that important additional information can be obtained from other standardized and recognised test procedures, and that such additional information may be essential to the closest possible evaluation of materials for use under particular conditions". (vide Ref. 5, under supplementary tests).

9.2. Studies with the Shrinkage Limit test.

With a view to finding out whether the volumetric changes of Black Cotton Soils disclose a possibility of comparing results, 37 soils from Hyderabad were subjected to the shrinkage limit test. The results are given in Table 11 and are plotted in Fig. 10 (according to Ref. 7, page 73). The range covered by the black cotton soils extends over the groups A-6 and A-7 of the U. S. P. R. A. System as shown in Fig. 10. It is thus seen that this test does not give conclusive results, because all the black cotton soils are definitely found to belong to the A-7 group. Further investigation of the shrinkage limit was, therefore, not done.

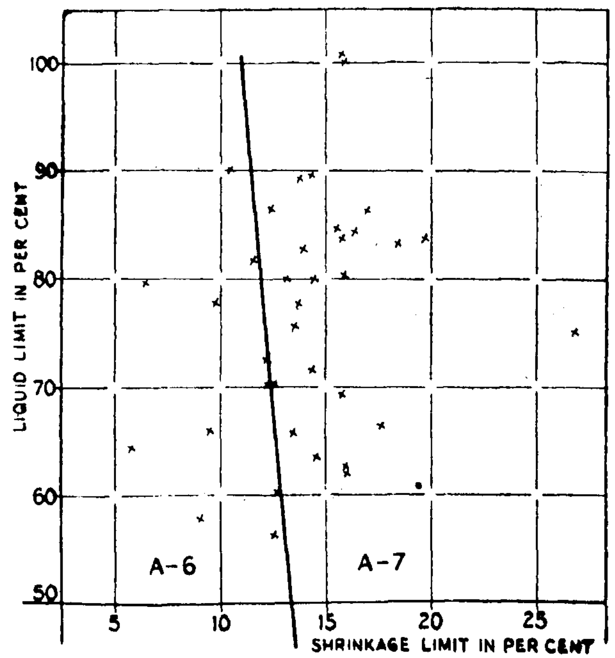
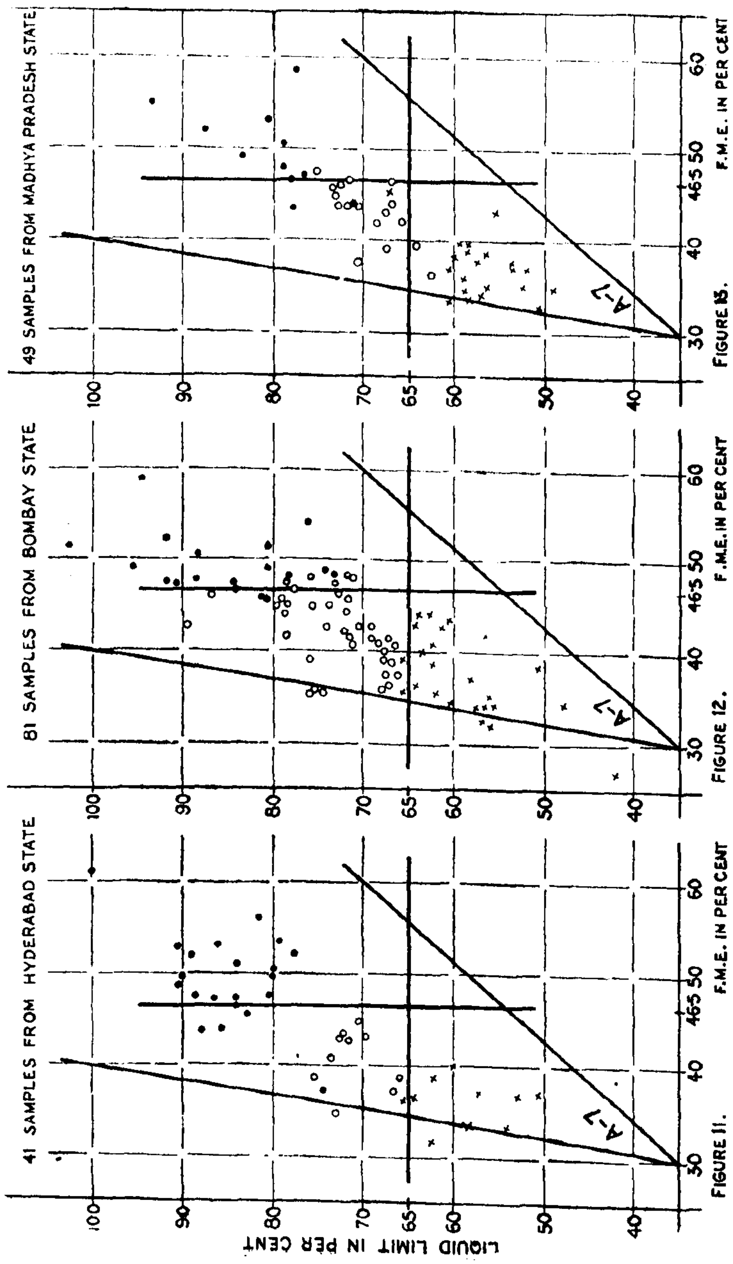


FIGURE 10. RELATION BETWEEN LIQUID LIMIT AND SHRINKAGE LIMIT OF 37 SAMPLES FROM HYDERABAD STATE SHOWN AFTER REF. 8 PAGE 73.



RELATION BETWEEN LIQUID LIMIT, FIELD MOISTURE EQUIVALENT (SHOWN AFTER REF. 8 PAGE 73) AND THE GROUP INDICES 20-30 (×), 30-40 (○) AND 40-50 (•) OF THE INVESTIGATED SOILS.

This can be taken as a confirmation of the results arrived at by Allen and reproduced in page 73 of Ref. 7 where, in the graph: Shrinkage Limit/Liquid Limit, no limits were given to separate the A-7 group.

9.3. Studies with the Field Moisture Equivalent test.

9.3.1. With 171 samples of black cotton soils, 41 from Hyderabad State, 81 from Bombay State and 49 from Madhya Pradesh, the Field Moisture Equivalent was determined and plotted against the liquid limit, as per Table 12, 13 and 14 and Figures 11, 12 and 13 (vide Ref. 7, page 73), using the same symbols according to their proposed sub-groups as was done in the Figures 6, 7, and 8. (group A-7a with crosses, group A-7b with hollow circles and group A-7c with dots). The results show that all the soils lie within the group A-7.

9.3.2. When, in all the three graphs, a horizontal line is plotted at liquid limit = 65, it is seen that the group A-7a crosses gets separated out from the rest. Thereafter, when a dividing line is drawn vertically at the most probable place, it divides the two remaining groups A-7b and A-7c (hollow circles and dots), with negligible exceptions. It is seen that this line lies near the value of field moisture equivalent of 46.5. This way of dividing the three sub-groups is correct for all the three graphs given in the Figures 11, 12 and 13. For any deviation in the results some margin can be left, but this does not alter the principle that the field moisture equivalent/liquid limit chart could be taken as one of the confirmative tests.

9.4. Studies with the Hygroscopic Moisture Absorption test.

9.4.1. The physical behaviour of the clayey soils in the field depends upon the amount of clay present and, therefore, upon the Hygroscopic Moisture Absorption. The determination of the Hygroscopic Moisture Absorption does not form one of the tests referred to as recognised in the U. S. P. R. A. specification, but its results are also of interest even though the test is not usually conducted.

9.4.2. The Hygroscopic Moisture Absorption of 173 black cotton soils (40 samples from Hyderabad, 84 from Bombay, and 49 from Madhya Pradesh) was determined by the static method of constant humidity*. The results are recorded in Tables 12, 13 and 14.

The values of the Hygroscopic Moisture Absorption in per cent are plotted against the Liquid Limit in Figure 14 for the soils from Hyderabad State, in Figure 15 for the soils from Bombay State and in Figure 16 for the soils from Madhya Pradesh.

9.4.3. The results were plotted in the three graphs according to their sub-group indexes 20–30, 30–40 and 40–50 respectively and were marked similarly : group A-7a with crosses, groups A-7b with hollow circles and A-7c with dots. As was done previously, a horizontal line at liquid limit 65 was plotted and it was seen that it divided almost without exception, the group indexes 20–30 (crosses) from those of 30–40 and 40–50. Furthermore, a dividing line was drawn vertically between the group indexes 30–40 and 40–50 (hollow circles and dots) which divided the two remaining sub-groups with a few exceptions. Compared with Figures 6, 7 and 8 as well as Figures 11, 12 and 13 (Field Moisture Equivalent Liquid Limit), we find that, in principle, the same system can be applied. The vertical parting line lies at about 17.5 per cent Hygroscopic Moisture Absorption for the soils from Hyderabad State and Madhya Pradesh, and at about 18·75 per cent for the soils from Bombay State in the three Figures 14, 15, and 16. The remark concerning the amount of error as mentioned before in para 8.4, which may possibly shift the horizontal and vertical parting lines, applies here also.

9.4.4. The Hygroscopic Moisture Absorption tests performed on these 173 soils give, in principle, the same results as those arrived at by using the two extended charts for group indexes ; the Field Moisture Equivalent tests of 171 soils support the results. It is found that the extended calculation for group indexes can be considered as confirmed and, therefore, valid and correct.

* Five grams of soil, powdered to pass the sieve ASTM No. 40, was dried in an oven at 105–107°C for 24 hours.

The sample was put into a weighing bottle of 1½ in. diameter and 1 in. height. The soil in the open bottle was kept in a dessicator over a solution of normal sulphuric acid to create a humidity of 98 per cent. To provide this constant humidity at different room temperatures, a vacuum of 10–15 m/m (Mercury) had to be produced in the dessicator.

After some days in the dessicator, the weighing bottle with the soil was weighed and subsequently on every alternate day until the weight was constant. The maximum moisture absorbed was calculated on the weight of dry soil.

For general information vide Ref. 6.

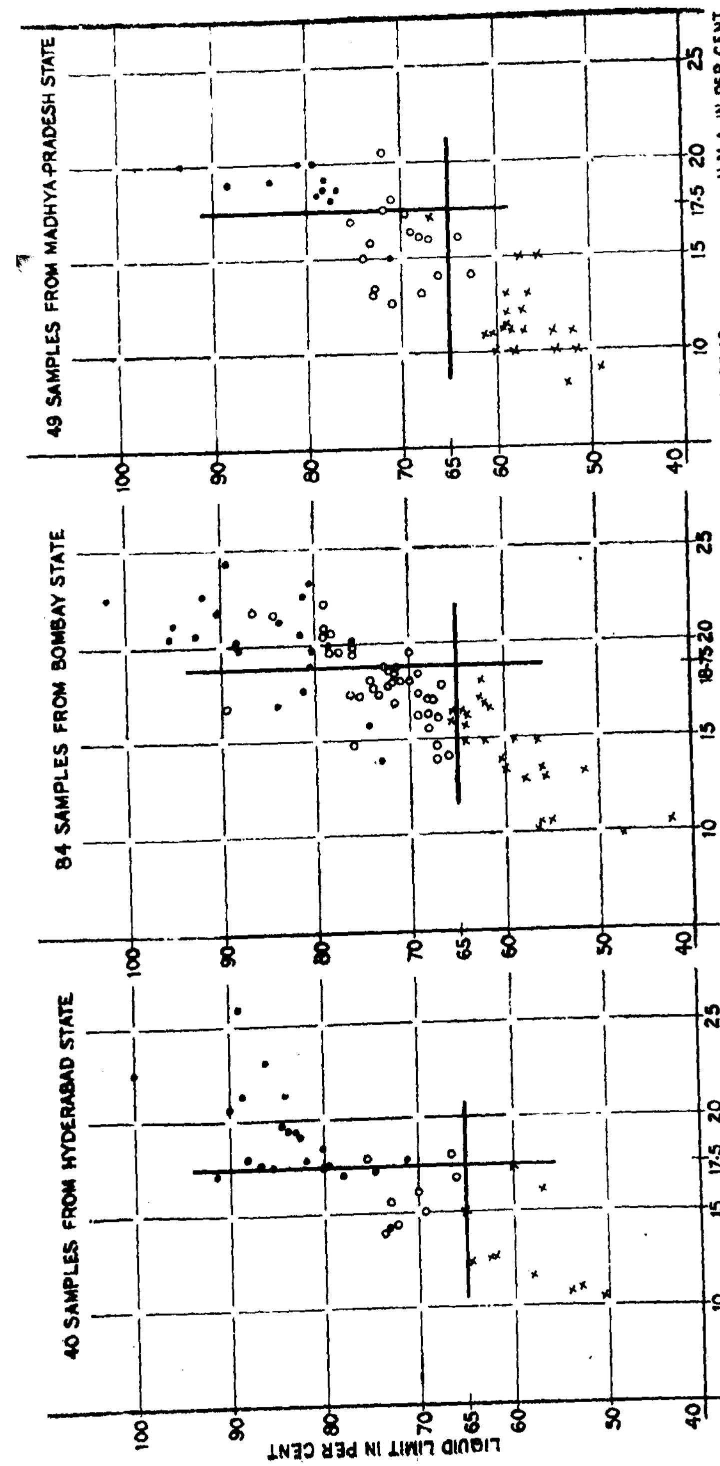


FIGURE 14. FIGURE 15. FIGURE 16.

RELATION BETWEEN LIQUID LIMIT, HYGROSCOPIC MOISTURE ABSORPTION AND THE GROUP INDICES 20–30 (×), 30–40 (○) AND 40–50 (•) OF THE INVESTIGATED SOILS.

10. DELIBERATION

10.1. As the liquid limit and the plastic limit are determined with the fraction of a soil passing A.S.T.M. Sieve No. 40, they are not influenced by the coarser part, the sandy fraction of the material retained on that sieve. It is, therefore, possible that soils with a similar fine fraction as the Indian Black Cotton Soil get classified as belonging to one of the three sub-groups: A-7a, A-7b or A-7c. But as these three sub-groups are characterised by high group indexes (20 to 50), no soil, which has a considerable coarse fraction, will fall in these sub-groups, because, in the calculation of the group indexes, the entire fines are involved (passing or retained on A.S.T.M. Sieve No. 200).

10.2. For stabilisation, the aim is to bring down the group index to below 16 for soils having such high liquid and plastic limits. If the Black Cotton Soils can be classified into narrow sub-groups, it might be possible to stabilise them more individually although they may be considered to belong to a comprehensive main group. This could contribute to their being utilised economically as construction materials by stabilisation.

11. SUMMARY

11.1. A large number of Black Cotton Soil samples from India have been analysed. As a result of the physical analysis it can be seen that the black cotton soils of India belong, in general, to the A-7 group of the 'U. S. Public Roads Administration Classification of 1945.'

11.2. A method for the sub-division of the A-7 group—the extension of the group indexes from 20 to 50—has been worked out and it has been shown that the heavy clay soils prevailing in India (Black Cotton Soils) can be divided into three sub-groups according to their group indexes, liquid limits, and plasticity indexes.

11.3. Tests with Field Moisture Equivalent and Hygroscopic Moisture Absorption confirm the addition of the three sub-groups to the main group A-7.

11.4. Further investigations have to be conducted to find methods of converting these soils economically into construction materials. Such investigation may be easier because the soils can now be classified more closely.

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TABLE 1

CHEMICAL ANALYSIS OF BLACK COTTON SOILS FROM HYDERABAD.

Serial No.	Sample No.	Total soluble solids Per cent	$CaCO_3$ Per cent	m.e.*/100 gm soil		pH	Organic carbon Per cent
				Exchangable Ca	Exchangable Na+K		
1	1	0.282	15.00	22.30	8.40	8.60	0.78
2	2	0.116		47.10	3.60	9.10	0.36
3	3	0.204	8.90	42.70	2.40	8.50	0.30
4	4	0.228	7.75	43.30	3.00	8.50	0.24
5	6	0.160	1.55	58.00	2.20	9.10	0.54
6	7	0.152	13.05	38.20	3.80	8.90	0.26
7	8	0.152	5.85	42.00	1.00	7.90	0.29
8	9	0.200	6.10	41.70	3.00	8.20	0.38
9	10	0.118	10.00	38.50	1.20	8.20	0.36
10	11	0.260	4.05	61.20	1.60	8.00	0.61
11	14	0.246	9.10	45.90	2.80	8.15	0.72
12	15	0.180	7.45	57.90	2.00	8.20	0.69
13	16	0.116	7.45	58.20	2.00	7.95	0.60
14	17	0.918	8.60	25.50	7.60	8.00	0.75
15	18	0.114		69.10	0.20	9.20	0.93
16	19	0.200	7.00	58.90	1.40	7.85	0.45
17	20	0.228	3.20	60.90	1.80	7.60	0.39
18	21	0.182	4.90	61.20	2.00	7.70	0.60
19	22	0.156	1.10	56.20	0.40	7.70	0.57
20	23	0.282	1.85	59.10	2.40	7.60	0.36
21	24	0.266	1.00	37.30	1.20	7.40	0.57
22	25	0.378	3.45	47.50	2.60	7.60	0.36
23	26	0.170	1.45	52.50	1.80	7.40	0.99
24	27	0.216	—	63.10	1.40	8.60	0.51
25	28	0.480	4.95	46.70	1.40	8.50	0.54
26	30	0.278	2.00	39.70	2.40	7.80	0.45
27	31	0.146	2.50	44.40	1.20	7.60	0.33
28	32	0.196	4.50	35.10	2.00	7.50	0.51
29	34	0.126	2.60	32.70	1.60	7.50	0.75
30	35	0.288	3.35	23.60	6.40	8.00	0.60
31	36	0.200	1.60	32.40	3.20	7.60	0.36
32	37	0.204	6.05	34.80	5.80	8.60	0.30
33	40	0.148	—	21.70	1.20	7.40	0.93
34	41	0.160	1.25	49.80	1.60	8.30	0.30
35	42	0.190	0.50	42.70	1.20	7.30	0.48
36	43	0.164	2.25	25.30	1.40	7.90	0.51
37	45	0.154	2.50	56.40	2.00	7.80	0.36

* m.e.=Milliequivalents.

Table 1—*Contd.*

Serial No.	Sample No.	Total soluble solids Per cent	$CaCo_3$ Per cent	m.e.*/100 gm soil		pH	Organic carbon Per cent
				Exchangable Ca	Exchangable Na+K		
38	46	0.210	5.55	45.50	4.80	8.30	0.51
39	49	0.176	10.40	48.50	2.40	7.90	0.45
40	50	0.240	4.85	45.70	4.80	8.20	0.57
41	51	0.254	...	56.20	1.80	8.20	. .
42	52	0.700	5.15	43.20	6.40	8.20	0.54
43	53	0.242	5.25	45.60	4.00	8.90	0.39
44	54	0.234	1.95	27.10	3.00	8.70	0.27
45	55	0.378	2.85	42.00	4.40	8.00	0.51
46	56	0.302	—	46.80	3.60	8.40	0.45
47	57	0.280	3.85	35.50	2.60	7.90	0.36
48	58	0.168	3.40	44.40	4.40	7.90	0.27
49	59	0.160	8.20	53.40	2.00	8.60	0.42
50	60	0.116	—	6.90	1.20	7.60	0.45
51	62	0.124	3.60	32.00	1.40	8.80	0.27
52	64	0.114	3.20	36.30	0.40	7.80	0.48

* m.e. Milliequivalents.

Tables 2 and 3—On pages 267 and 268.

TABLE 4.

PHYSICAL PROPERTIES OF THE SOILS FROM HYDERABAD STATE.

Serial No.	Sample No.	Liquid Limit per cent	Plasticity Index	Fraction passing 200 sieve per cent
1	1	80.36	48.95	93.30
2	2	89.76	47.62	87.32
3	3	71.60	37.89	87.61
4	4	81.71	48.18	88.15
5	6	86.05	39.85	81.40
6	7	73.23	41.23	83.0
7	8	62.50	36.79	77.14
8	9	82.72	50.02	92.80
9	10	54.04	18.01	91.80
10	11	69.81	30.87	98.0
11	14	70.30	33.97	94.89
12	15	100.0	58.07	95.47
13	16	75.48	34.74	87.70
14	17	86.63	54.49	90.70
15	18	80.19	40.31	95.40
16	19	84.55	44.05	92.31
17	20	90.55	49.99	96.88
18	21	84.02	45.96	96.35
19	22	83.28	44.02	95.42
20	23	83.45	46.87	99.07
21	24	65.96	32.81	93.02
22	25	88.55	47.05	96.60
23	26	60.19	24.65	85.20
24	27	100.80	58.24	83.07
25	28	65.40	30.73	83.26
26	29	65.47	24.98	84.60
27	30	66.35	32.42	92.80
28	31	69.30	36.29	86.57
29	32	74.48	44.93	91.35
30	34	62.03	38.12	76.22
31	35	84.02	53.84	90.85
32	36	57.15	34.53	77.87
33	37	89.22	55.22	89.91
34	39	49.60	29.38	79.65
35	40	49.51	26.01	75.80
36	41	77.41	46.29	85.35
37	42	64.46	36.22	69.73
38	43	58.14	31.89	68.54
39	45	85.49	54.13	95.35
40	46	90.17	51.86	92.67
41	47	85.59	47.39	86.17

Table 4—*Contd.*

Serial No.	Sample No.	Liquid Limit per cent	Plasticity Index	Fraction Passing 200 sieve per cent
42	49	72.19	42.00	84.32
43	50	91.35	58.52	87.13
44	51	79.54	41.90	90.81
45	52	77.76	43.03	92.55
46	53	83.42	46.49	83.80
47	54	56.54	34.85	63.30
48	55	80.14	44.82	87.10
49	56	87.94	54.34	87.87
50	57	73.43	42.00	89.70
51	58	72.58	39.87	91.0
52	59	79.68	41.67	88.75
53	60	63.57	30.96	69.20
54	62	53.07	27.77	75.30
55	64	51.11	29.98	77.0
56	65	58.43	29.06	82.26

Table 5.

PHYSICAL PROPERTIES OF THE SOILS FROM BOMBAY STATE.

Serial No.	Sample No.	Liquid Limit per cent	Plasticity Index	Fraction passing 200 sieve Per cent
1	66	42.31	22.87	83.19
2	69	47.76	25.93	85.80
3	71	55.92	30.59	68.17
4	72	58.80	27.49	81.88
5	73	51.47	22.28	80.20
6	74	67.61	41.25	90.80
7	75	64.05	23.71	80.71
8	76	62.18	26.67	94.20
9	77	81.01	40.59	97.10
10	78	73.92	37.31	87.32
11	79	81.20	50.95	93.30
12	80	62.62	27.9	72.86
13	81	56.23	30.62	87.46
14	82	73.28	45.92	85.81
15	85	88.38	49.86	93.60
16	87	65.63	27.21	91.55
17	88	76.32	42.65	95.62
18	89	80.44	42.42	97.31
19	90	75.63	39.81	87.79
20	91	66.57	31.79	95.73
21	92	88.34	49.93	89.16
22	93	72.22	39.56	90.30
23	94	79.13	35.14	95.97
24	95	81.69	41.51	91.60
25	96	75.92	35.92	83.08
26	97	89.43	46.88	97.62
27	98	67.65	29.78	94.11
28	100	68.80	31.76	94.80
29	102	65.37	29.06	70.94
30	103	91.74	54.35	97.51
31	104	70.96	29.95	94.70
32	105	84.00	41.82	94.90
33	106	64.02	31.76	80.87
34	107	95.36	53.98	98.29
35	108	83.83	42.58	94.34
36	109	57.63	24.13	89.56
37	110	72.59	32.47	96.10
38	111	74.06	43.76	96.26
39	112	66.06	32.10	91.70
40	113	78.40	37.01	93.22

Table 5—*Contd.*

Serial No.	Sample No.	Liquid Limit per cent	Plasticity Index	Fraction passing 200 sieve per cent
41	114	73.19	41.09	86.70
42	115	89.84	37 39	77.40
43	116	78.40	37.01	93.22
44	117	83.88	43.56	90.00
45	118	75.85	29.53	84.00
46	119	67.86	39.50	89.70
47	120	71.88	40.38	88.80
48	121	67.16	32.02	91.80
49	122	80.50	42.80	95.15
50	123	71.43	39.03	90.45
51	124	61.64	25.64	84.95
52	125	62.39	23.72	91.15
53	126	75.96	36.43	93.00
54	128	78.68	41.00	85.00
55	130	94.95	48.92	93.05
56	132	101.88	51.88	93.50
57	133	69.20	32.20	85.75
58	134	69.98	34.27	90.00
59	136	64.38	30.17	77.13
60	137	80.07	43.67	87.60
61	138	78.84	33.43	91.90
62	139	74.14	40.46	91.80
63	140	69.15	26.65	94.70
64	141	56.37	27.37	84.40
65	143	60.09	25.19	88.60
66	144	55.15	23.45	81.30
67	145	67.00	33.10	90.00
68	146	66.90	28.00	96.00
69	148	56.87	31.77	76.80
70	149	60.12	28.42	95.60
71	150	90.09	60.79	86.90
72	151	56.72	28.15	90.00
73	152	70.31	39.44	81.50
74	153	76.42	40.84	89.65
75	154	63.52	28.23	83.05
76	155	92.44	52.29	94.25
77	156	79.42	34.01	88.90
78	157	71.05	34.31	88.15
79	158	71.17	30.36	86.00
80	159	86.74	36.74	83.40
81	160	77.51	36.01	93.20
82	161	72.07	38.67	82.00
83	162	62.32	30.42	89.35
84	163	71.16	33.36	86.35
85	164	71.59	39.99	89.75
86	170	67.44	31.17	96.25

TABLE 6.

PHYSICAL PROPERTIES OF THE SOILS FROM MADHYA PRADESH STATE.

Serial No.	Sample No.	Liquid Limit per cent	Plasticity Index	Fraction passing 200 sieve per cent
1	172	66.08	32.90	94.40
2	173	67.04	42.79	73.46
3	174	50.06	29.99	85.40
4	175	60.44	31.56	82.60
5	176	61.44	33.36	83.60
6	177	53.30	30.51	77.10
7	178	58.49	28.64	94.31
8	179	56.24	33.96	87.50
9	180	72.19	41.35	92.49
10	181	73.10	37.39	98.15
11	182	66.37	38.10	95.00
12	183	49.22	28.2	89.38
13	184	65.98	38.89	96.09
14	185	60.76	32.01	85.05
15	186	60.29	29.45	81.10
16	187	50.58	28.13	73.53
17	188	49.18	25.46	66.97
18	189	61.60	28.62	71.00
19	191	66.82	34.88	97.00
20	193	71.83	35.40	89.82
21	194	81.00	42.0	93.78
22	195	73.40	34.04	90.72
23	196	67.37	27.65	93.74
24	197	79.32	47.32	91.88
25	198	79.47	45.51	91.60
26	199	50.65	26.40	84.08
27	200	56.64	24.01	78.91
28	201	67.07	39.34	92.39
29	202	52.06	29.99	79.28
30	203	75.58	36.58	83.76
31	204	58.85	32.91	81.54
32	205	67.21	35.99	87.10
33	206	71.27	42.14	91.66
34	208	53.49	31.27	94.14
35	209	52.61	27.80	92.70
36	210	73.96	36.06	91.49
37	211	53.84	25.27	94.10
38	212	49.33	26.38	97.42
39	213	58.52	27.12	94.30
40	214	58.74	31.03	92.60

Table 6—*Contd.*

Serial No.	Sample No.	Liquid Limit per cent	Plasticity Index	Fraction passing 200 sieve per cent
41	215	56.86	31.16	89.20
42	216	66.03	34.76	90.36
43	217	59.44	32.34	86.91
44	218	58.42	32.01	85.00
45	219	60.16	32.47	86.76
46	222	72.82	37.16	91.91
47	223	83.81	43.71	93.01
48	224	71.18	37.16	82.38
49	227	64.19	38.34	86.04
50	228	67.91	32.10	95.09
51	229	68.87	41.25	83.54
52	230	72.19	37.32	84.95
53	231	55.62	27.48	79.80
54	232	57.43	26.25	96.90
55	233	76.80	44.54	87.13
56	234	57.30	31.40	91.63
57	235	62.52	30.42	91.13
58	236	88.14	48.14	89.16
59	237	93.38	46.72	91.12
60	238	73.05	42.42	84.99
61	239	70.88	42.69	81.07
62	240	60.57	29.97	79.53
63	242	59.09	30.59	92.25
64	243	60.05	33.29	83.04
65	244	67.68	39.97	84.47
66	245	77.55	45.02	97.33
67	246	78.27	43.76	94.45
68	247	78.68	50.63	93.20

TABLE 7

BLACK COTTON SOILS WITH GROUP INDEX 20-30, SUBGROUP A-7a.

Serial No.	Sample No.	State	Liquid Limit (per cent)	Plasticity Index	Fraction passing 200 (per cent)	Group Index
1	8	Hyderabad	62.50	36.79	77.14	29
2	10	,,	54.04	18.01	91.80	20
3	26	,,	60.19	24.65	85.20	25
4	28	,,	65.40	30.73	83.26	28
5	29	,,	65.47	24.98	84.60	25
6	34	,,	62.03	38.12	76.22	29
7	36	,,	57.15	34.53	77.87	27
8	39	,,	49.60	29.38	79.65	22
9	40	,,	49.51	26.01	75.80	20
10	42	,,	64.46	36.22	69.73	26
11	43	,,	58.14	31.89	68.54	22
12	54	,,	56.54	34.85	63.30	20
13	60	,,	63.57	30.96	69.20	22
14	62	,,	53.07	27.77	75.30	22
15	64	,,	51.11	29.98	77.00	24
16	65	,,	58.43	29.06	82.26	26
17	66	Bombay	42.31	22.87	83.19	18
18	69	,,	47.76	25.93	85.80	22
19	71	,,	55.92	30.59	68.17	20
20	72	,,	58.80	27.49	81.88	25
21	73	,,	51.47	22.28	80.20	19
22	75	,,	64.05	23.71	80.71	23
23	76	,,	62.18	26.67	94.20	29
24	80	,,	62.62	27.90	72.86	22
25	81	,,	56.23	30.62	87.46	28
26	87	,,	65.63	27.21	91.55	29
27	102	,,	65.37	29.06	70.94	22
28	106	,,	64.02	31.76	80.87	29
29	109	,,	57.63	24.13	89.56	24
30	124	,,	61.64	25.64	84.92	25
31	125	,,	62.39	23.72	91.15	26
32	136	,,	64.38	30.17	77.13	26
33	141	,,	56.37	27.37	84.40	25
34	143	,,	60.09	25.10	88.60	25
35	144	,,	55.15	23.45	81.30	21
36	148	,,	56.87	31.77	76.80	25
37	149	,,	60.12	28.42	95.60	30
38	151	,,	56.72	28.15	90.00	27
39	154	,,	63.52	28.53	83.05	27
40	162	,,	62.32	30.42	89.35	29
41	174	Madhya Pradesh	50.06	29.99	85.40	27

Table 7—*Contd.*

Serial No.	Sample No.	State	Liquid Limit (per cent)	Plasticity Index	Fraction passing 200 (per cent)	Group Index
42	175	Madhya Pradesh	60.44	31.56	82.68	28
43	176	,,	61.44	33.36	83.60	30
44	177	,,	53.30	30.51	77.10	23
45	178	,,	58.49	28.64	94.31	29
46	179	,,	59.24	33.96	87.50	29
47	183	,,	49.22	28.20	89.38	25
48	185	,,	60.76	32.01	85.05	29
49	186	,,	60.29	29.54	81.10	26
50	187	,,	50.58	28.13	73.58	20
51	188	,,	49.18	25.46	66.97	17
52	189	,,	61.60	28.62	71.00	22
53	196	,,	67.37	27.65	93.74	30
54	199	,,	50.65	26.40	84.08	23
55	200	,,	56.64	24.01	78.91	22
56	202	,,	52.06	29.99	79.28	24
57	204	,,	58.85	32.91	81.54	28
58	208	,,	53.49	31.27	94.14	29
59	209	,,	52.61	27.80	92.70	27
60	211	,,	53.84	25.27	94.10	26
61	212	,,	49.33	26.38	97.42	25
62	213	,,	58.52	27.12	94.30	29
63	214	,,	58.74	31.03	92.60	30
64	215	,,	56.86	31.16	89.20	29
65	217	,,	59.44	32.34	86.91	29
66	218	,,	58.42	32.01	85.00	30
67	219	,,	60.16	32.47	86.47	30
68	231	,,	55.62	27.42	79.80	23
69	232	,,	57.43	26.25	96.90	28
70	234	,,	57.30	31.40	91.63	30
71	240	,,	60.57	29.97	79.53	26
72	242	,,	59.05	30.59	92.25	30
73	243	,,	60.05	33.29	83.04	29

TABLE 8

BLACK COTTON SOILS WITH GROUP INDEX 30-40, SUBGROUP A-7b.

Serial No.	Sample No.	State	Liquid Limit (per cent)	Plasticity Index	Fraction passing 200 (per cent)	Group Index
1	3	Hyderabad	71.60	37.89	87.61	37
2	6	,,	86.05	39.85	81.40	38
3	7	,,	73.23	41.23	83.00	38
4	11	,,	69.81	30.87	98.00	35
5	14	,,	70.30	33.97	94.89	35
6	16	,,	75.48	34.74	87.70	35
7	24	,,	65.96	32.81	93.02	34
8	30	,,	66.35	32.42	92.80	34
9	31	,,	69.30	36.29	86.56	34
10	49	,,	72.19	42.19	84.32	38
11	57	,,	73.43	42.00	89.70	40
12	58	,,	72.58	39.87	91.00	40
13	74	Bombay	67.61	41.25	90.80	38
14	78	,,	73.92	37.31	87.32	36
15	90	,,	75.63	39.81	87.79	39
16	91	,,	66.57	31.79	95.73	34
17	93	,,	72.22	39.56	90.30	39
18	94	,,	79.13	35.14	95.97	40
19	96	,,	75.92	35.92	83.08	35
20	98	,,	67.65	29.78	94.11	33
21	100	,,	68.80	31.76	94.80	34
22	104	,,	70.96	29.95	94.70	34
23	110	,,	72.59	32.47	96.10	36
24	112	,,	66.06	32.16	91.70	32
25	114	,,	73.19	41.09	86.70	40
26	115	,,	89.84	37.39	77.40	32
27	116	,,	78.40	37.01	93.22	40
28	118	,,	75.85	29.53	84.00	32
29	119	,,	67.86	39.50	89.70	37
30	120	,,	71.88	40.38	88.80	39
31	121	,,	67.16	32.02	91.80	33
32	123	,,	71.43	39.03	90.45	38
33	126	,,	75.96	36.43	93.00	39
34	128	,,	78.68	41.00	85.00	39
35	133	,,	69.20	32.20	85.75	31
36	134	,,	69.98	34.27	90.00	34
37	138	,,	78.84	33.43	91.90	37
38	139	,,	74.14	40.46	91.80	40
39	140	,,	69.15	26.65	94.70	32
40	145	,,	67.00	33.10	90.00	33

Table 8—*Contd.*

Sr. No.	Sample No.	State	Liquid Limit (per cent)	Plasticity Index	Fraction passing 200 (per cent)	Group Index
41	146	Bombay	66.90	28.00	96.00	32
42	152	,,	70.30	39.44	81.50	35
43	153	,,	76.42	40.84	89.65	40
44	156	,,	79.42	34.01	88.90	37
45	157	,,	71.05	34.31	88.15	34
46	158	,,	71.17	30.36	86.00	31
47	159	,,	86.74	36.74	83.40	37
48	160	,,	77.51	36.01	93.20	39
49	161	,,	72.07	38.67	82.00	35
50	163	,,	71.16	33.36	86.35	32
51	164	,,	71.59	39.99	89.75	39
52	170	,,	67.44	31.54	96.25	35
53	172	Madhya Pradesh	66.08	32.90	94.40	34
54	173	,,	67.04	42.79	73.46	34
55	180	,,	72.19	41.35	92.49	40
56	181	,,	73.10	37.39	98.15	40
57	182	,,	66.37	38.10	95.00	38
58	184	,,	65.98	38.89	96.09	39
59	191	,,	66.82	34.88	97.00	37
60	193	,,	71.83	35.40	89.82	36
61	195	,,	73.40	34.04	90.72	36
62	201	,,	67.07	39.34	92.39	38
63	203	,,	75.58	36.58	83.76	36
64	205	,,	67.21	35.99	87.10	34
65	210	,,	73.96	36.06	91.49	37
66	216	,,	66.03	34.76	90.36	34
67	222	,,	72.82	37.16	91.91	38
68	224	,,	71.18	37.16	82.38	34
69	227	,,	64.19	38.34	86.04	34
70	228	,,	67.91	32.10	95.00	35
71	229	,,	68.87	41.25	83.54	37
72	230	,,	72.19	37.32	84.95	36
73	235	,,	62.52	30.42	91.13	31
74	238	,,	73.05	42.42	84.90	39
75	239	,,	70.88	42.69	81.07	37
76	244	,,	67.68	39.97	84.47	36

TABLE 9.

BLACK COTTON SOILS WITH GROUP INDEX 40-50, SUBGROUP A-7c.

Serial No.	Sample No.	State	Liquid Limit (per cent)	Plasticity Index	Fraction passing 200 (per cent)	Group Index
1	1	Hyderabad	80.36	48.95	93.30	45
2	2	,,	89.76	47.62	87.32	44
3	4	,,	81.71	48.18	88.15	44
4	9	,,	82.72	50.02	92.80	46
5	15	,,	100.00	58.07	95.47	48
6	17	,,	86.63	54.49	90.70	46
7	18	,,	80.19	40.31	95.40	43
8	19	,,	84.55	44.05	92.31	46
9	20	,,	90.55	49.99	96.88	48
10	21	,,	84.02	45.96	96.35	46
11	22	,,	83.28	44.02	95.42	46
12	23	,,	83.45	46.87	99.07	49
13	25	,,	88.55	47.05	96.60	48
14	27	,,	100.80	58.24	83.07	43
15	32	,,	74.48	44.93	91.35	42
16	35	,,	84.02	53.84	90.85	45
17	37	,,	89.22	55.22	89.61	45
18	41	,,	77.41	46.29	85.35	42
19	45	,,	85.49	54.13	95.35	48
20	46	,,	90.17	51.86	92.67	46
21	47	,,	85.59	47.39	86.17	43
22	50	,,	91.35	58.52	87.13	44
23	51	,,	79.54	41.90	90.81	41
24	52	,,	77.76	43.03	92.55	43
25	53	,,	83.42	46.49	83.80	43
26	55	,,	80.14	44.82	87.10	43
27	56	,,	87.94	54.34	87.87	44
28	59	,,	79.68	41.67	88.75	41
29	77	Bombay	81.01	40.59	97.10	44
30	79	,,	81.20	50.95	93.30	46
31	82	,,	73.28	45.92	93.60	44
32	85	,,	88.38	49.86	91.41	46
33	88	,,	76.32	42.65	95.62	44
34	89	,,	80.44	42.42	97.31	45
35	92	,,	88.34	49.93	89.16	45
36	95	,,	81.69	41.51	91.60	43
37	97	,,	89.43	46.88	97.62	48
38	103	,,	91.74	54.35	97.51	48
39	105	,,	84.00	41.82	91.90	44
40	107	,,	95.36	53.98	98.29	49
41	108	Bombay	83.83	42.58	94.34	46
42	111	,,	74.06	43.76	96.26	44
43	113	,,	78.40	37.01	93.22	40
44	117	,,	83.88	43.56	90.00	44
45	122	,,	80.50	42.80	95.15	45
46	130	,,	94.95	48.92	93.05	47
47	132	,,	101.88	51.88	93.50	47
48	137	,,	80.07	43.67	87.60	43
49	150	,,	90.09	60.79	86.90	44
50	155	,,	92.44	52.29	94.25	47
51	194	M. Pradesh	81.00	42.00	91.78	45
52	197	,,	79.32	47.32	91.88	44
53	198	,,	79.47	45.51	91.60	44
54	206	,,	71.27	42.14	91.60	41
55	223	,,	83.80	43.71	93.09	46
56	233	,,	76.88	44.54	87.13	42
57	236	,,	88.14	48.14	89.16	45
58	237	,,	93.38	46.72	91.12	46
59	245	,,	77.55	45.02	97.33	46
60	246	,,	78.27	43.76	94.45	45
61	247	,,	78.68	50.63	93.20	45

TABLE 9—*Contd.*

Table 10—on page 280

TABLE 11

SHRINKAGE LIMIT AND LIQUID LIMIT OF 37 BLACK COTTON SOILS FROM HYDERABAD STATE.

Serial No.	Sample No.	Liquid Limit per cent	Shrinkage Limit per cent
1	1	80.36	16.07
2	2	89.76	14.38
3	3	71.60	14.44
4	4	81.70	11.66
5	6	86.05	17.12
6	8	62.50	15.97
7	9	82.72	13.85
8	11	69.81	12.60
9	14	70.30	12.43
10	15	100.00	15.90
11	16	75.48	13.53
12	17	86.63	12.44
13	18	80.19	13.37
14	19	84.55	15.58
15	22	83.28	18.46
16	23	83.45	15.79
17	24	65.96	9.49
18	26	60.19	12.25
19	27	100.80	15.80
20	28	65.40	13.50
21	30	66.35	17.69
22	31	69.33	15.86
23	32	74.48	26.84
24	34	62.03	16.15
25	35	84.02	17.30
26	37	89.22	13.74
27	41	77.41	13.48
28	42	64.46	5.76
29	43	58.14	9.13
30	46	90.17	10.40
31	51	79.54	6.51
32	52	77.76	9.80
33	53	83.42	19.75
34	54	56.54	12.43
35	55	80.14	14.56
36	58	72.58	12.21
37	60	63.57	14.65

TABLE 12

FIELD MOISTURE EQUIVALENT & HYGROSCOPIC MOISTURE ABSORPTION OF BLACK COTTON SOILS FROM HYDERABAD.

Serial No.	Sample No.	Liquid Limit percent	Field Moisture Equivalent per cent	Hygroscopic Moisture Absorption per cent
1	1	80.36	47.63	18.48
2	3	71.60	42.59	14.79
3	4	81.71	54.00	17.87
4	6	86.05	53.00	23.15
5	7	73.23	34.33	14.20
6	8	62.55	31.76	12.44
7	9	82.72	45.45	19.16
8	10	54.04	33.50	10.59
9	11	69.81	43.14	16.07
10	14	70.38	44.93	—
11	15	100.00	61.30	22.57
12	16	75.48	38.43	17.80
13	17	86.63	47.95	17.6
14	18	80.19	49.80	—
15	19	84.55	51.00	19.76
16	20	90.55	52.81	—
17	21	84.02	47.33	21.30
18	22	83.28	—	19.32
19	24	65.96	38.56	16.85
20	25	88.55	47.45	21.37
21	26	60.19	40.07	17.29
22	28	65.40	35.98	14.87
23	30	66.35	37.17	17.84
24	31	69.33	—	14.98
25	32	74.48	37.33	16.76
26	34	62.03	38.65	12.43
27	35	84.02	46.46	19.49
28	36	57.15	37.08	16.05
29	37	89.24	52.06	25.79
30	42	64.46	36.55	12.25
31	43	58.14	33.77	11.62
32	45	85.49	44.14	17.24
33	46	90.17	49.87	20.56
34	49	72.19	43.07	14.32
35	50	91 35	48.75	17.07
36	52	77.76	52.00	17.12
37	55	80.14	50.45	17.44
38	56	87.94	43.75	17.90
39	57	73.43	40.50	14.17
40	58	72.58	43.18	15.53
41	59	79.68	53.40	17.47
42	62	53.07	37.60	10.78
43	64	51.11	37.01	10.36

Table 13

FIELD MOISTURE EQUIVALENT & HYGROSCOPIC MOISTURE ABSORPTION OF BLACK COTTON SOILS FROM BOMBAY.

Serial No.	Sample No.	Liquid Limit Per cent.	Field Moisture Equivalent Per cent	Hygroscopic Moisture Absorption Per cent
1	66	42.31	26.72	10.50
2	69	47.76	34.35	9.77
3	71	55.92	32.07	12.93
4	72	58.80	37.17	14.90
5	73	51.47	38.16	13.13
6	74	67.61	37.43	16.27
7	75	64.05	35.96	15.58
8	76	62.18	40.27	16.96
9	77	81.01	—	22.50
10	78	73.92	45.16	17.63
11	79	81.20	45.31	17.54
12	80	62.62	43.88	18.15
13	81	56.23	31.95	13.66
14	82	73.28	48.46	13.91
15	85	88.38	50.08	19.84
16	87	65.63	35.43	15.86
17	88	76.32	53.71	20.04
18	89	80.44	48.70	18.80
19	90	75.63	44.80	17.21
20	91	66.57	40.54	17.85
21	92	88.34	47.96	20.09
22	93	72.22	42.21	17.88
23	94	79.13	45.65	21.92
24	95	81.69	—	20.50
25	96	75.92	38.99	19.99
26	97	89.43	—	24.36
27	98	67.65	39.24	17.12
28	100	68.80	40.68	17.36
29	102	65.37	38.80	16.39
30	103	91.74	52.40	22.79
31	106	64.02	44.18	14.90
32	107	95.36	48.41	20.62
33	108	83.83	46.58	17.15
34	109	57.63	33.40	12.68
35	110	72.59	46.33	18.76
36	111	74.06	48.54	15.67
37	112	66.06	38.15	13.88
38	113	78.40	47.67	19.99
39	114	73.19	46.82	17.19
40	115	89.84	42.55	16.65

Table 13—*Contd.*

Serial No.	Sample No.	Liquid Limit	Field Moisture Equivalent Per cent	Hygroscopic Moisture Absorption Per cent
41	116	78.40	47.67	19.48
42	117	83.88	46.94	21.22
43	118	75.85	47.88	14.70
44	119	67.86	35.72	15.46
45	120	71.88	43.96	17.90
46	121	67.16	41.37	15.84
47	122	80.50	51.10	23.29
48	123	71.43	41.34	16.71
49	124	61.64	42.14	16.47
50	125	62.39	38.47	16.73
51	126	75.96	35.26	19.50
52	128	78.68	44.95	20.49
53	130	94.95	59.20	21.15
54	132	101.88	50.96	22.24
55	133	69.20	42.44	16.31
56	134	69.98	41.44	17.99
57	136	64.38	42.96	16.33
58	137	80.07	46.10	19.68
59	138	78.84	44.06	20.22
60	139	74.14	42.50	18.02
61	140	69.15	41.10	18.36
62	141	56.37	33.61	10.47
63	143	60.09	34.51	13.20
64	144	55.15	33.86	10.57
65	145	67.00	36.38	14.35
66	146	66.90	38.70	13.87
67	148	56.87	41.88	10.05
68	149	60.12	43.38	13.86
69	150	90.09	46.84	21.84
70	151	56.72	35.14	14.70
71	152	70.30	42.55	19.52
72	153	76.42	36.12	17.38
73	154	63.52	40.00	16.12
74	155	92.44	47.36	20.58
75	156	79.42	45.17	20.50
76	157	71.05	40.46	17.96
77	158	71.17	47.74	18.13
78	159	86.74	46.22	21.69
79	160	77.51	46.50	19.59
80	161	72.07	35.47	18.49
81	162	62.32	35.46	14.37
82	163	71.10	47.91	18.36
83	164	71.59	45.54	18.42
84	170	67.44	40.20	16.99

TABLE 14

FIELD MOISTURE EQUIVALENT & HYGROSCOPIC MOISTURE ABSORPTION OF BLACK COTTON SOILS FROM MADHYA PRADESH

Serial No.	Sample No.	Liquid Limit Per cent	Field Moisture Equivalent Per cent	Hygroscopic Moisture Absorption Per cent
1	193	71.83	46.47	20.38
2	194	81.00	52.79	19.93
3	195	73.40	44.92	16.57
4	196	67.37	45.43	17.50
5	197	79.32	47.71	18.29
6	198	79.47	50.61	19.77
7	199	50.65	33.00	10.18
8	200	56.64	38.59	12.97
9	201	67.07	43.95	16.35
10	202	52.06	37.11	11.20
11	203	75.58	47.55	16.48
12	204	58.85	38.71	11.60
13	205	67.21	47.19	15.77
14	206	71.27	44.13	14.62
15	208	53.49	37.58	10.00
16	209	52.61	34.87	8.42
17	210	73.96	45.49	14.87
18	211	53.84	37.17	11.10
19	212	49.33	34.97	8.87
20	213	58.52	33.47	11.39
21	214	58.74	35.84	11.29
22	215	56.86	35.15	10.77
23	216	66.03	42.00	13.67
24	217	59.44	39.79	13.18
25	218	58.42	39.40	10.04
26	219	60.16	38.09	10.02
27	222	72.82	46.31	13.39
28	223	83.80	49.39	19.09
29	224	71.18	43.88	17.90
30	227	64.19	39.68	15.62
31	228	67.91	43.27	15.63
32	229	68.87	42.39	16.55
33	230	72.19	43.80	17.58
34	231	55.62	43.31	14.82
35	232	57.43	37.74	15.05
36	233	76.80	47.30	18.60
37	234	57.30	33.88	12.27
38	235	62.52	36.28	14.29
39	236	88.14	52.15	18.98
40	237	93.38	55.00	20.10
41	238	73.05	43.70	13.43
42	239	70.88	37.51	12.51
43	240	60.57	33.33	11.06
44	242	59.09	34.47	11.96
45	243	60.65	36.79	11.19
46	244	67.68	38.99	12.68
47	245	77.55	58.56	18.15
48	246	78.27	46.42	18.60
49	427	78.63	43.57	18.53

Plate I

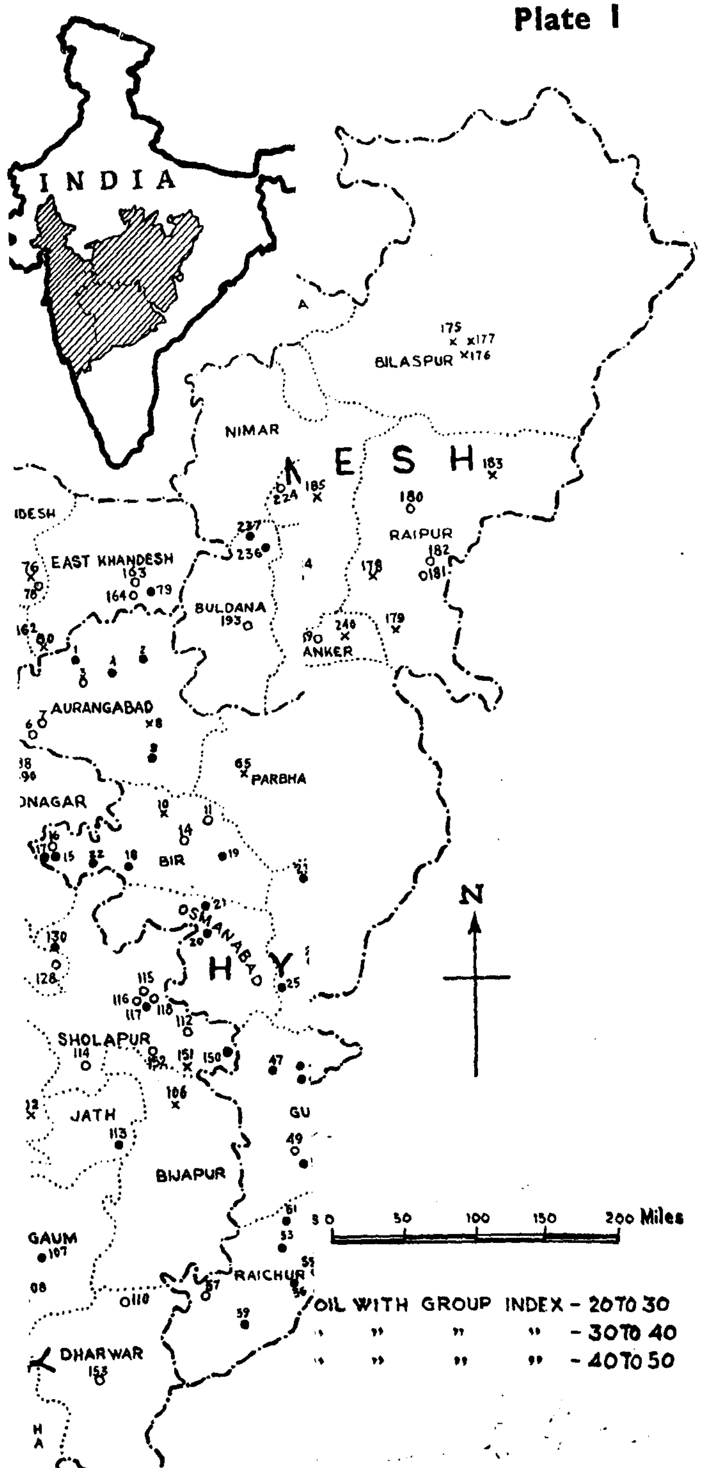


PLATE I. TTON SOILS ARE TAKEN

Plate I

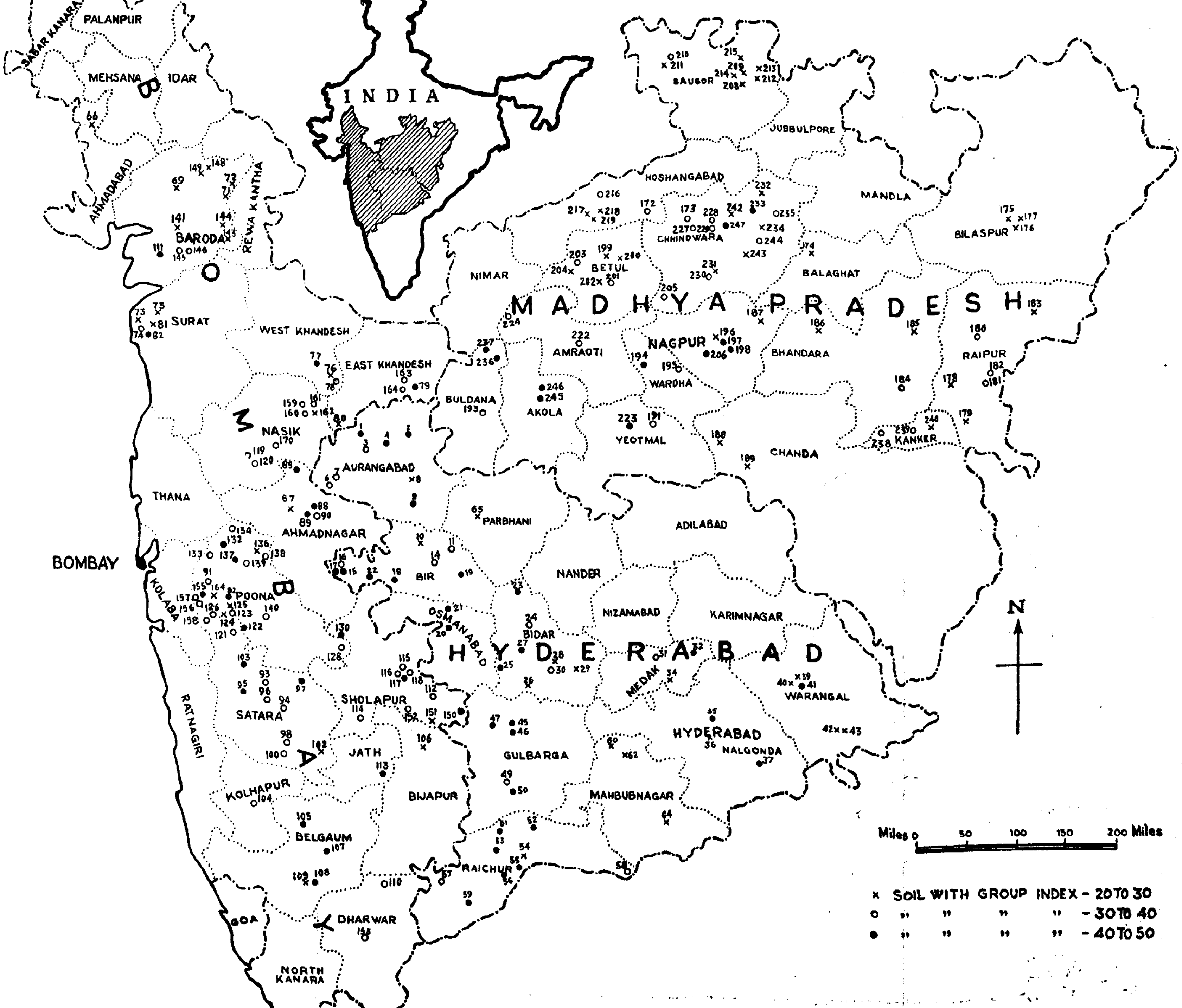


PLATE 1. LOCALITY WHERE THE 210 BLACK COTTON SOILS ARE TAK

BSY-750

THICK STUFF-BITUMEN!

Here's a familiar sight—this gentleman is pouring Bitumen on to a road surface to bind and strengthen it. It is no exaggeration to say that the economy of the country as a whole depends upon good roads. When Bitumen is used in their construction they can withstand present day traffic and monsoon rains.

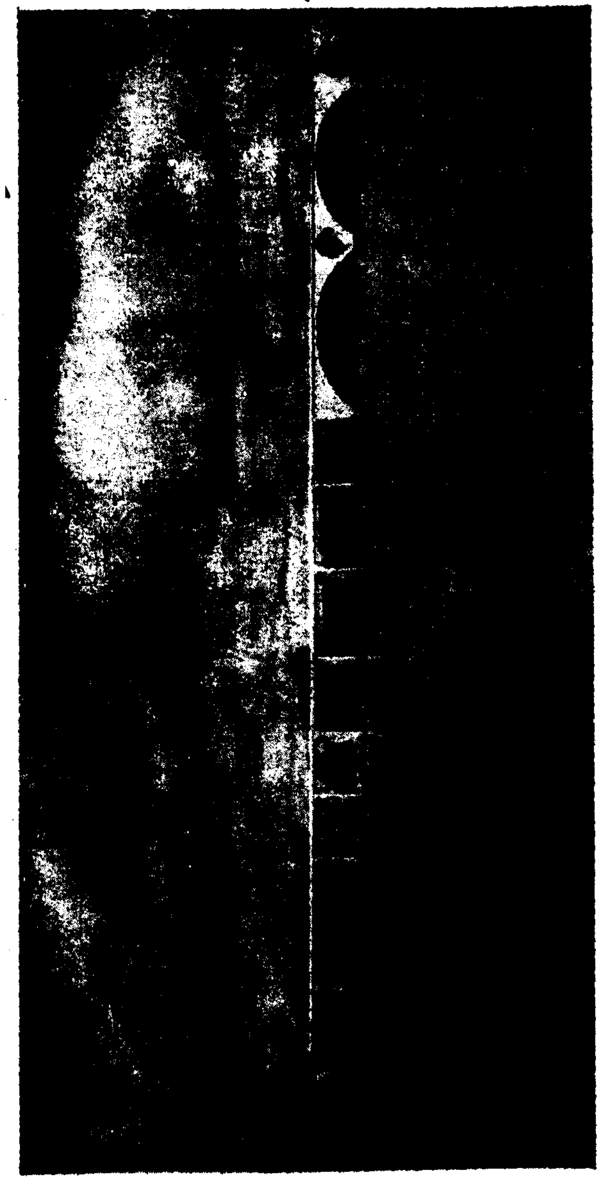
• • • •

For irrigation schemes too, Bitumen is important as it is used for canal linings to prevent seepage and corrosion. It is also used to build airfield runways, to insulate electrical cables, for damp-proof courses and for refrigeration purposes.

• • • •

So, next time you are driving along a well-kept road, trying to overtake a heavily-laden lorry or a train of bullock carts, console yourself with the thought that it is really the road surface that's taking most of the strain. Think of Bitumen, and think of Burmah-Shell.

BURMAH-SHELL...IN INDIA'S LIFE AND PART OF IT



RAIND BRIDGE—Constructed near Ambikapur, Surguja, this bridge has a total length of 598 feet consisting of R.C.C. rigid frame spans varying from 31 feet to 48 feet each and of filled spandrel R.C.C. barrel arches of 110 feet span each. The arch foundations are up to 20 feet deep below the bed.

J. C. GAMMON LIMITED

HAMILTON HOUSE BALLARD ESTATE BOMBAY-1