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SHEAR FAILURE IN ANISOTROPIC MATERIALS

By

C. G. SWAMINATHAN, B.A., B.E. (Hons.), C.E. (Hons.), M.Sc. (Eng.), D.I.C. (Lond.), A.M.I.H.E. (Lond.), A.M.I.E. (Ind.)

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SYNOPSIS

Extending the principles laid down by Casagrande and Carillo[1] and modifying the version given by Hank and McCarty[2], this Paper deals with the problem "Given the shear constants on two mutually perpendicular planes and one of the principal stresses, to determine the critical circle of failure in an anisotropic material."

Both analytical and graphical solutions are given. As the procedure given by Barber[2] involves a certain amount of trial and error, a purely-graphical solution is worked out for the general case. Twelve particular cases have also been studied and graphical procedure for some special cases included.

A discussion based on the various equations obtained is also incorporated, from which it follows that the values of test data are likely to be misinterpreted unless the consideration of anisotropy is stressed and the validity of Coulomb's Law is also ascertained.

1. INTRODUCTION

1.1. While evaluating the primary physical properties of a soil or granular mass at shear failure, it is usually assumed that the material is isotropic and that the values of cohesion and internal friction remain the same in any plane between two given principal planes. Based on these assumptions, a series of critical circles (Mohr's stress circles of failure) are drawn for given values of the principal stresses and from the slope and intercept of the common tangent to the circles, the angle of friction and cohesion of the material are obtained. But the behaviour of some plastic soils and asphalt mixtures under test makes one doubt the validity of assumption of isotropy. Further, in an asphaltic concrete mixture, it is found that the orientation of the different particles and aggregates to the vertical axis varies from plane to plane. So the possibility of non-isotropic nature of the material, as regards shear characteristics arises.

1.2. The foundation for the study of the mechanics of the shear failure in anisotropic materials has been laid by Casagrande and Carillo[1] and by Hank and McCarty[2]. The former have enunciated the basic principles for purely cohesive and for purely friction materials, and the latter have extended these principles to include materials possessing both cohesion and friction. Further, only a trial and error method of graphical analysis was suggested by the former, while the latter have developed a more effective graphical method by which the critical circle could be drawn and the shear strength on the failure plane could be evaluated.

1.3. The graphical method of Hank and McCarty could be used only if both the maximum and minimum shear strength across two mutually perpendicular planes are given. Both these two principal strengths would not be known, unless both the principal stresses at failure were also known. Hence, if only one of the principal strengths is given, the graphical procedure as given by Hank and McCarty[2] could not be adopted. Also, as per this procedure though both the shear strengths on the two principal planes are known, the shear constants on those planes could not be determined with a single critical circle.

1.4. So the problem is "Given the shear constants on two mutually perpendicular planes and one of the principal stresses, to determine, the critical circle of failure and to obtain the other principal stress and the slope of the failure plane". This obviously presents a more general picture of the anisotropic material and the problem becomes more definite. As the mathematical analysis for this case is a bit cumbersome to use, a graphical solution seems desirable. As an attempt towards this, Barber[3] has given a non-Euclidean method. But this method suffers from the fact that it is one of trial and error.

1.5. Hence in this Paper, it is attempted to develop a purely graphical solution for the most general case of an anisotropic material, by which it will be possible to draw the circle of failure more decisively and to obtain the slope of the failure plane etc. without any trial and error.

2. ASSUMPTIONS

2.1. Before the mathematical analysis and grapical methods are given, the assumptions involved are stated below. It may be mentioned that these assumptions are the same as those given in references 1 and 2.

(a) The normal and shear stresses on any plane at the time of failure are related to each other according to Coulomb's Law.

(b) The same relationship with varying values of the parameters holds good for all planes through a given point including the principal planes.

(c) The shear strength on any plane, in between the two principal planes, obeys a straight line law, in other words, the shear strength line for the critical circle is a straight line tangential to the circle.

(d) The principal shear strengths for induced anisotropic materials are developed on the planes of the principal stresses—maximum shear over the plane of major principal stress and minor shear on the plane of minor principal stress.

3. MATHEMATICAL ANALYSIS

3.1. Notation

τ = Shear strength component on the plane of failure.

σ = Normal stress acting prependicular to the plane of failure.

K = Cohesion, independent of the normal stress on the plane of failure.

ϕ = Angle of internal friction.

τ_1 = Shear strength on the major principal plane.

τ_3 = Shear strength on the minor principal plane.

σ_1 = Major principal stress.

σ_3 = Minor principal stress.

K_1 = Cohesion on the plane of major principal stress.

K_3 = Cohesion on the plane of minor principal stress.

ϕ_1 = Angle of internal friction on the plane of major principal stress.

ϕ_3 = Angle of internal friction on the plane of minor principal stress.

θ = Angle that the plane of failure makes with the plane of major principal stress.

h, f, g, c, μ and λ = Constants. See Fig 1.

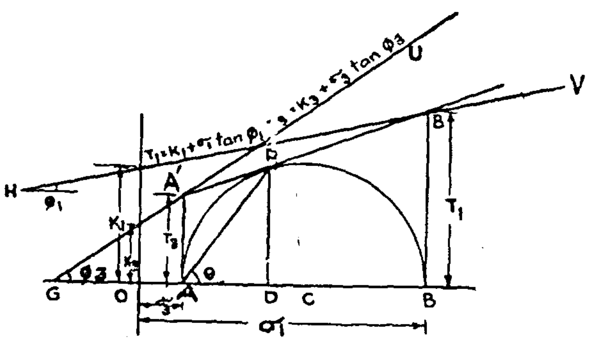


FIG-1 CRITICAL CIRCLE FOR ANISOTROPIC MATERIAL

3.2. General Equations

Based on the assumptions mentioned in para. 2.1. (a) and (b), and Coulomb's Law

$$\tau_1 = K_1 + \sigma_1 \tan \phi_1 \qquad \dots (1)$$

$$\tau_3 = K_3 + \sigma_3 \tan \phi_3 \qquad \dots (2)$$

Based on assumptions mentioned in para. 2.1 (c) and (d), in accordance with the findings of Hank and McCarty[1].

$$\sigma_1 - \sigma_3 = 2\sqrt{\tau_1 \; \tau_3} \qquad \dots (3)$$

$$\tan \theta = \sqrt{\frac{\tau_1}{\tau_3}} \qquad \dots (4)$$

Given ϕ_1, ϕ_3 ; K_1, K_3, the shear constants on the two principal planes and σ_3, to determine the other 3 unknowns namely σ_1, τ_1, and τ_3. τ_3 is obtained from Equation (2). σ_1 and τ_1 will have to be obtained with the help of Equations (1) & (3).

3.3. Equation for the Centre of the Critical Circle

The centre of the critical circle will have the co-ordinates

$$= \left\{ \sigma_3 + \frac{\sigma_1 - \sigma_3}{2} \right\}, 0$$

The radius of the critical circle is $= \dfrac{\sigma_1 - \sigma_3}{2}$

Since, $\tau_1 = K_1 + \sigma_1 \tan \phi_1$ and $\sigma_1 - \sigma_3 = 2\sqrt{\tau_1 \; \tau_3}$

$$(\sigma_1 - \sigma_3)^2 = 4\tau_3 (K_1 + \sigma_1 \tan \phi_1)$$

(i.e.) $\sigma_1^2 - 2\sigma_1 (\sigma_3 + 2\tau_3 \tan \phi_1) + (\sigma_3^2 - 4\tau_3 K_1) = 0$

(i.e.) $\sigma_1 = (\sigma_3 + 2\tau_3 \tan \phi_1) \pm$

$$2\sqrt{K_1 \tau_3 + \tau_3 \sigma_3 \tan \phi_1 + \tau_3^2 \tan^2 \phi_1} \qquad (5a)$$

Based on assumption (4), taking the $+^{ve}$ value,

$$\frac{\sigma_1 - \sigma_3}{2} = \tau_3 \tan \phi_1 +$$

$$\sqrt{\tau_3 \{K_1 + (\sigma_3 + \tau_3 \tan \phi_1) \tan \phi_1\}} \qquad \dots (5)$$

3.4. Equation for the Major Principal Stresses at Failure

Expressing τ_3 in Equation (5), in terms of $(\sigma_3 \tan \phi_3 + K_3)$, it follows that,

$$\frac{\sigma_1 - \sigma_3}{2} = (K_3 + \sigma_3 \tan \phi_3) \tan \phi_1 +$$

$$\sqrt{(K_3 + \sigma_3 \tan \phi_3)\{K_1 + \sigma_3 \tan \phi_1 + \tan^2 \phi_1 (K_3 + \sigma_3 \tan \phi_3)\}}$$

(i.e.) $\dfrac{\sigma_1 - \sigma_3}{2} = (K_3 + \sigma_3 \tan \phi_3) \tan \phi_1 +$

$$\sqrt{(\sigma_3 \tan \phi_3 + K_3)\{\sigma_3 \tan \phi_1 (1 + \tan \phi_1 \tan \phi_3) + (K_1 + K_3 \tan^2 \phi_1)\}} \qquad \dots (6)$$

(i.e.) $\sigma_1 = 2 K_3 \tan \phi_1 + (1 + 2 \tan \phi_1 \tan \phi_3)\, \sigma_3$

$$+ 2\sqrt{(\sigma_3 \tan \phi_3 + K_3)\{\sigma_3 \tan \phi_1 (1 + \tan \phi_1 \tan \phi_3) + (K_1 + K_3 \tan^2 \phi_1)} \dots (7)$$

3.5. Equation for the Angle of the Plane of Failure

$$\text{Now, } \tan\theta = \sqrt{\frac{\tau_1}{\tau_3}} = \frac{\sqrt{\tau_1\,\tau_3}}{\tau_3} = \frac{\sigma_1 - \sigma_3}{2\tau_3}$$

$$\text{(i.e.) } \tan\theta = \frac{\sigma_1 - \sigma_3}{2(K_3 + \sigma_3 \tan\phi_3)}$$

Taking the values of $\dfrac{\sigma_1 - \sigma_3}{2}$ as found in Equation (6)

$$\tan\theta = \frac{K_3 \tan\phi_1 + \sigma_3 \tan\phi_1 \tan\phi_3}{K_3 + \sigma_3 \tan\phi_3} + \frac{\sqrt{(K_3+\sigma_3 \tan\phi_3)\{\sigma_3 \tan\phi_1(1+\tan\phi_1 \tan\phi_3)+K_1+K_3 \tan^2\phi_1\}}}{K_3+\sigma_3 \tan\phi_3}$$

$$\text{(i.e.) } \tan\theta = \tan\phi_1 + \sqrt{\tan^2\phi_1 + \frac{K_1 + \sigma_3 \tan\phi_1}{K_3 + \sigma_3 \tan\phi_3}} \quad \ldots \quad (8)$$

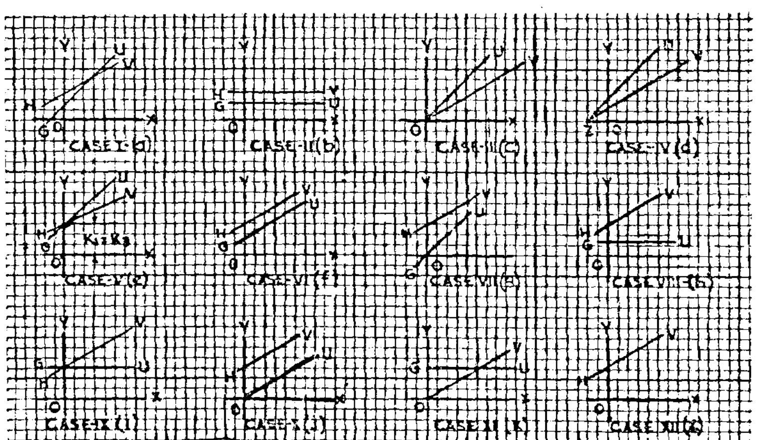


FIG-2 DIFFERENT CASES OF COULOMB'S LINES
U for K_3 & ϕ_3, V for K_1 & ϕ
HV REPRESENTS $T_1 = K_1 + \sigma_1 \tan\phi_1$
GU REPRESENTS $T_3 = K_3 + \sigma_3 \tan\phi_3$

3.6. Particular Cases

Case I. The general case $K_1 \neq K_3 \neq o$; $\phi_1 \neq \phi_3 \neq o$
See Fig. 2(a)—Equations have been given above.

Case II. For a purely cohesive material $\phi_1 = \phi_3 = o$
But $K_1 \neq K_3 \neq o$ See Fig. 2(b)

Equation (7) becomes

$$\sigma_1 = \sigma_3 + 2\sqrt{K_1\,K_3}$$

This shows that the difference between the principal stresses is always a constant and that the radius of the critical circle is also always a constant. Also equation (8) becomes $\tan\theta = \sqrt{\dfrac{K_1}{K_3}}$ which shows that the slope of the failure plane is also always a constant, irrespective of the magnitude of the principal stresses.

Case III. For a purely friction material when $K_1 = K_3 = o$
But, $\phi_1 \neq \phi_3 \neq o$ See Fig. 2(c)

Equation (7) becomes

$$\sigma_1 = \sigma_3 \{(1 + 2\tan\phi_1 \tan\phi_3) + 2\sqrt{\tan\phi_1 \tan\phi_3 (1 + \tan\phi_1 \tan\phi_3)}\}$$

which shows that the relationship between the principal stresses will be linear.

$$\text{Also, } \tan\theta = \tan\phi_1 \left\{ 1 + \sqrt{1 + \frac{1}{\tan\phi_1 \tan\phi_3}} \right\}$$

which shows clearly that the failure plane will always make a constant angle with the plane of major principal stress, irrespective of the magnitudes of the principal stresses.

Case IV. For a material for which the two Coulomb's lines for strength on the principal planes intersect the X-axis at the same point i.e., $\phi_1 \neq \phi_3 \neq o$; $K_1 \neq K_3 \neq o$; But, $K_1 \tan\phi_3 = K_3 \tan\phi_1$. See Fig. 2 (d)

Equation (7) becomes

$$\sigma_1=\sigma_3\ \{1+2\tan\phi_1\tan\phi_3+2\sqrt{\tan\phi_1\tan\phi_3(1+\tan\phi_1\tan\phi_3)}\}$$
$$+2K_3\tan\phi_1\left\{1+\sqrt{1+\frac{1}{\tan\phi_1\ \tan\phi_3}}\right\}$$

which shows that a linear relationship between the principal stresses exists.

$$\text{Also, }\tan\theta=\tan\phi_1\left\{1+\sqrt{1+\frac{1}{\tan\phi_1\tan\phi_3}}\right\}$$

which clearly shows that the failure plane will always make a constant angle with the plane of major principal stress, irrespective of the magnitude of the principal stresses. Also it can be seen that this constant is the same as for case III.

Case V. For a material possessing the same cohesion but varying angles of internal friction on the principal planes $\phi_1 \neq \phi_3 \neq o$; $K_1 = K_3 \neq o$ See Fig. 2 (e)

Equation (7) becomes

$$\sigma_1=2K_3\tan\phi_1+\sigma_3(1+2\tan\phi_1\tan\phi_3)+$$
$$2\sqrt{(\sigma_3\tan\phi_3+K_3)\{\sigma_3\tan\phi_1(1+\tan\phi_1\tan\phi_3)+K_3(1+\tan^2\phi_1)\}}$$

which shows that the relationship between the principal stresses is not a simple linear one.

Also,

$$\tan\theta=\tan\phi_1+\sqrt{\tan^2\phi_1+\frac{K_3+\sigma_3\tan\phi_1}{K_3+\sigma_3\tan\phi_3}}$$

which shows that for different values of the minor principal stress, the slope of the failure plane varies.

Case VI. For a material possessing the same angle of internal friction but unequal cohesion on the principal planes i.e., $K_1 \neq K_3 \neq o$; But, $\phi_1 = \phi_3 \neq o$ See Fig. 2 (f)

Equation (7) becomes

$$\sigma_1=2K_3\tan\phi_1+(1+2\tan^2\phi_1)\sigma_3+$$
$$2\sqrt{K_3+\sigma_3\tan\phi_1)\{\sigma_3\tan\phi_1(1+\tan^2\phi_1)+(K_1+K_3\tan^2\phi_1)\}}$$

which shows that the relationship between the principal stresses at failure is not always linear.

Also,

$$\tan\theta=\tan\phi_1+\sqrt{\tan^2\phi_1+\frac{K_1+\sigma_3\tan\phi_1}{K_3+\sigma_3\tan\phi_1}}$$

which shows that for different values of the minor principal stress the slope of the failure plane will vary.

Case VII. For a material for which there is no cohesion on the minor principal plane $K_3 = o$; $\phi_1 \neq \phi_3 \neq o$ See Fig. 2 (g)

Equation (7) becomes

$$\sigma_1=(1+2\tan\phi_1\tan\phi_3)\sigma_3+$$
$$2\sqrt{\{\sigma_3\tan\phi_1(1+\tan\phi_1,\tan\phi_3)+K_1\}\sigma_3\tan\phi_3}$$

If $K_1 = o$ instead of K_3; $\sigma_1 = 2K_3\tan\phi_1 +$
$$(1+2\tan\phi_1\tan\phi_3)\sigma_3$$
$$+2\sqrt{(\sigma_3\tan\phi_3+K_3)\{\sigma_3\tan\phi_1(1+\tan\phi_1\tan\phi_3)+K_3\tan^2\phi_1\}}$$

which shows that the relationship between the principal stresses is not a simple linear one.

$$\text{Also when }K_3=o;\ \tan\theta=\tan\phi_1+\sqrt{\tan^2\phi_1+\frac{K_1+\sigma_3\tan\phi_1}{\sigma_3\tan\phi_3}}$$

If $K_1 = o$ instead of K_3; $\tan\theta = \tan\phi_1 +$
$$\sqrt{\tan^2\phi_1+\frac{\sigma_3\tan\phi_1}{K_3+\sigma_3\tan\phi_3}}$$

The slope of the failure plane changes with the magnitude of the minor principal stress.

Case VIII. For a material which has no frictional resistance but only cohesion on the minor principal plane $\phi_3 = o$; $K_1 \neq K_3 \neq o$ See Fig. 2(h)

Equation (7) becomes

If $\phi_3=o$; $\sigma_1=\sigma_3+2\sqrt{\{\sigma_3\tan\phi_1+(K_1+K_3\tan^2\phi)\}K_3}+2K_3\tan\phi_1$

If $\phi_1=o$, instead of ϕ_3; $\sigma_1=\sigma_3+2\sqrt{(\sigma_3\tan\phi_3+K_3)K_1}$

which show that the relationship between the principal stresses is not always linear.

Also, when $\phi_3 = o$;

$$\tan\theta = \tan\phi_1 + \sqrt{\tan^2\phi_1 + \frac{K_1 + \sigma_3 \tan\phi_1}{K_3}}$$

If $\phi_1 = o$ instead of ϕ_3; $\tan\theta = \sqrt{\dfrac{K_1}{K_3 + \sigma_3 \tan\phi_3}}$

which clearly show the variation of the slope of the failure plane with the minor principal stress.

Case IX. For a material possessing no frictional resistance on the minor principal plane, and the same cohesion on the major and minor principal planes. i.e., $K_1 = K_3 \neq o$; $\phi_3 = o$. See Fig. 2(i)

Equation (7) becomes

When $\phi_3 = o$; $\sigma_1 = 2K_3 \tan\phi_1 + \sigma_3 +$

$$2\sqrt{(\sigma_3 \tan\phi_1 + K_3 + K_3 \tan^2\phi_1)\,K_3}$$

If $\phi_1 = o$ instead of ϕ_3; $\sigma_1 = \sigma_3 + 2\sqrt{(\sigma_3 \tan\phi_3 + K_3)\,K_3}$

which show that the relationship between the principal streeses is not simply linear.

Also, when $\phi_3 = o$,

$$\tan\theta = \tan\phi_1 + \sqrt{\tan^2\phi_1 + \frac{K_3 + \sigma_3 \tan\phi_1}{K_3}}$$

If $\phi_1 = o$ instead of ϕ_3; $\tan\theta = \sqrt{\dfrac{K_3}{K_3 + \sigma_3 \tan\phi_3}}$

which show that the angle of the failure plane varies for different values of the minor principal stress.

Case X. For a material having the same angle of internal friction on the principal planes and having no cohesion on the minor principal plane i.e., $\phi_1 = \phi_3 \neq o$; $K_3 = o$; See Fig. 2(j)

Equation (7) becomes

If $K_3 = o$; $\sigma_1 = \sigma_3 (1 + 2\tan^2\phi_1) +$

$$2\sqrt{\{\sigma_3 \tan\phi_1 (1 + \tan^2\phi_1) + K_1\}\,\sigma_3 \tan\phi_1}$$

If $K_1 = o$, instead of K_3;

$$\sigma_1 = 2K_3 \tan\phi_1 + \sigma_3 (1 + 2\tan^2\phi_1)$$
$$+2\sqrt{\{\sigma_3 \tan\phi_1 (1 + \tan^2\phi_1) + K_3 \tan^2\phi_1\}\{K_3 + \sigma_3 \tan\phi_3\}}$$

which show that the law is not simply that of a straight line.

Also, when $K_3 = o$, $\tan\theta = \tan\phi_1 + \sqrt{\tan^2\phi_1 + \dfrac{K_1 + \sigma_3 \tan\phi_1}{\sigma_3 \tan\phi_1}}$

If $K_1 = o$ instead of K_3; $\tan\theta = \tan\phi_1 +$

$$\sqrt{\tan^2\phi_1 + \frac{\sigma_3 \tan\phi_1}{K_3 + \sigma_3 \tan\phi_1}}$$

which show the variation of the slope of the failure plane with the value of the minor principal stress.

Case XI. For a material which has only frictional resistance in the major principal plane and pure cohesion on the minor principal plane $K_1 = o$; and $\phi_3 = o$. See Fig. 2(k)

Equation (7) becomes

If $K_1 = o$ & $\phi_3 = o$;

$$\sigma_1 = 2K_3 \tan\phi_1 + \sigma_3 + 2\sqrt{\{\sigma_3 \tan\phi_1 + K_3 \tan^2\phi_1\}\,K_3}$$

If $K_3 = o$ & $\phi_1 = o$; $\sigma_1 = \sigma_3 + 2\sqrt{K_1\,\sigma_3 \tan\phi_3}$

which give the law governing the relationship between the principal stresses.

Also, when $K_1 = o$ & $\phi_3 = o$;

$$\tan\theta = \tan\phi_1 + \sqrt{\tan^2\phi_1 + \frac{\sigma_3 \tan\phi_1}{K_3}}$$

When $K_3 = o$ & $\phi_1 = o$; $\tan\theta = \sqrt{\dfrac{K_1}{\sigma_3 \tan\phi_3}}$

which show that the slope of the failure plane varies with the value of the minor principal stress.

Case XII. For a material which possesses both friction and cohesion on the major principal plane only. $K_3 = o$ and $\phi_3 = o$ See Fig. 2(l)

Equation (7) becomes

$$\text{If } K_3 = o \ \&\ \phi_3 = o; \qquad \sigma_1 = \sigma_3$$

$$\text{If } K_1 = o \ \&\ \phi_1 = o; \qquad \sigma_1 = \sigma_3$$

which shows that the critical circle is a point.

Also,

$$\text{when } K_3 = o \ \&\ \phi_3 = o, \qquad \tan\theta = \infty.$$

$$\text{When } K_1 = o \ \&\ \phi_1 = o; \qquad \tan\theta = o$$

which show that the failure plane is parallel to one of the principal planes.

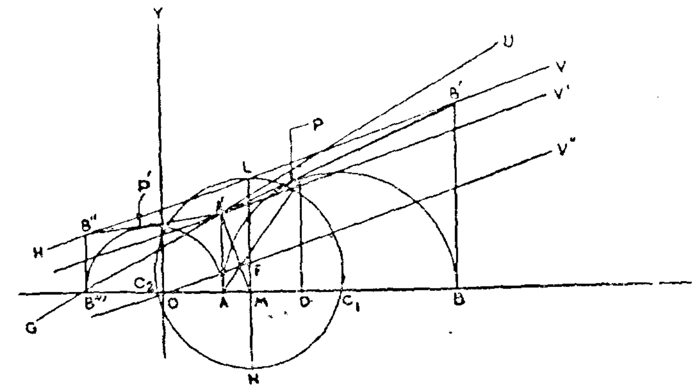
Finally, it may be recorded here that the interchange of the values of the parameters of Coulomb's Law does not affect the various relationships for cases I to VI. For cases VII to XII slight variations are noted when values for ϕ_1 instead of ϕ_3 or K_1 instead of K_3 or *vice versa* are incorporated. These variations have been noted and two sets of equations have been given for cases VII to XII.

Also, it may be seen from the various equations, that some cases are more or less similar. Taking for example cases III and IV, it can be seen that the modified equation (7) for the two cases represent two parallel lines, one passing through the origin (for case III) and the other passing through a given point on one of the axes (for case IV). Similarly cases VIII and IX are similar. So are cases VI and X. And the corresponding equations for such sets show that the corresponding curves representing those equations are similar in nature but with slight modifications, depending upon the initial conditions involved.

4. GRAPHICAL METHODS

4.1. General case: As stated in para. 3.2, the problem is to determine τ_1 and σ_1 for given values of ϕ_1, ϕ_3 ; K_1, K_3 and σ_3. Graphically, see Fig. 1, given, the line GU representing $\tau_3 = K_3 + \sigma_3 \tan\phi_3$, line HV representing $\tau_1 = K_1 + \sigma_1 \tan\phi_1$ and $OA = \sigma_3$, it is required to draw the critical circle of failure and to determine the plane of failure.

It is obvious that on erecting a perpendicular to the X-axis (A' being the point of intersection of this perpendicular and line GU), AA' gives τ_3 and the strength line touching the critical circle should pass through this point A'.



FIG·3. CONSTRUCTION FOR THE CRITICAL CIRCLE FOR THE GENERAL CASE
$K_1 \neq K_3 \neq 0$ & $\phi_1 \neq \phi_3 \neq 0$

Fig. 3 gives the construction for the critical circle. Through A' (in Fig. 3) a line $A'V'$ is drawn parallel to HV and through O, the origin, another line OV'' is also drawn parallel to HV. At A' a perpendicular AM is erected to $A'V'$ so as to cut the X-axis at M. Through M, a perpendicular line to the X-axis is drawn cutting OV'' at F and HV at L. This line LM is produced downwards to N such that $MN = AA'$. With L and N as the ends of a diameter, a circle is drawn cutting the X-axis at C_1 and C_2. With C_1 as centre and radius equal to C_1A, a circle is drawn so that it may cut the X-axis at B. This circle is the required critical circle.

At B, a perpendicular to the X-axis is erected to cut HV at B'. OB will give σ_1 and BB' will give τ_1. Joining $A'B'$ the strength line is obtained. This line will touch the critical circle at P and AP will be the plane of failure.

A similar procedure is adopted with C_2 as centre and C_2A as radius giving P', B'' and B'''.

Proof

$$C_1 A = C_1M + MA = \sqrt{LM.MN} + AA' \tan \angle MA'A$$

$$\text{So } C_1A = AA' \tan\phi_1 + \sqrt{AA'(LF+FM)}$$

$$= \tau_3 \tan\phi_1 + \sqrt{\tau_3\{K_1+(OA+AM)\tan\phi_1\}}$$

$$= \tau_3 \tan\phi_1 + \sqrt{\tau_3\{K_1+(\sigma_3+\tau_3\tan\phi_1)\tan\phi_1\}}$$

$$= \frac{\sigma_1-\sigma_3}{2}$$

This is the same as Equation (5) in para. 3.3.

Also,

$$C_2 A = \sqrt{\tau_3(K_1+\sigma_3\tan\phi_1+\tau_3\tan^2\phi_1)} - \tau_3\tan\phi_1$$

$$\text{(i.e.) } \frac{\sigma_3-\sigma_1}{2} = \tau_3\tan\phi_1 - \sqrt{\tau_3(K_1+\sigma_3\tan\phi_1+\sigma_1\tan^2\phi_1}$$

This represents the other value for $\frac{\sigma_1-\sigma_3}{2}$ in Equation (5 a) in para. 3.3.

A similar procedure can be adopted for all the cases mentioned in para, 3.6.

Particular Cases

4.2. Purely Friction Material

For a purely friction material, as in case III of para. 3.6, a modified graphical procedure is suggested. The equations for this case as given in para. 3.6 suggest that the value of σ_3 is not necessary for the determination of the slope of the failure planes and for given values of ϕ_1 and ϕ_3, θ is readily obtained.

Referring to Fig. 4, the slope of the line OV gives the angle ϕ_1 and the slope of OU gives ϕ_3. Any point S is taken on the X-axis and a perpendicular is erected cutting OU at U and OV at V. Through O a line at right angles to OU is drawn cutting the vertical line US produced at Q. On the line SV, a length $VR = VQ$ is marked off. With R and S as the ends of a diameter a circle is drawn cutting the horizontal line VT at T.

A length $VW = VT$ is marked off on SR. The slope of line OW gives the angle of the plane of failure.

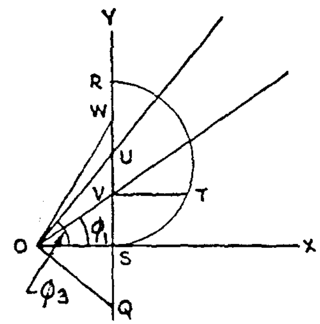


Fig. 4. **Construction for θ (K=O)**

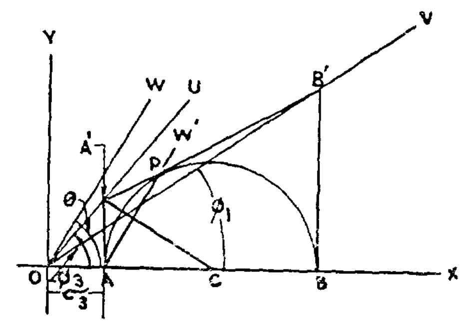


Fig. 5. **Construction for Critical Circle (K=O)**

A line OW is drawn in Figure 5 such that $\angle WOX = \theta$. In the Figure a point A is taken on the X-axis, giving $OA = \sigma_3$. A perpendicular to the X-axis at A is erected cutting OU (representing angle ϕ_3) at A'. Through A, a line AW' is drawn parallel to OW. From A', a perpendicular $A'C$ is dropped on this line AW' and cutting the X-axis at C. With C as centre and CA as radius a circle is drawn cutting the X-axis at B and AW' at P. This is the required circle of failure. Erecting a perpendicular at B and cutting OV (representing ϕ_1) at B' and on joining A' and B' it is found that $A'B'$ touches the critical circle at P. OB gives σ_1; BB' gives τ_1; AP is the failure plane and $A'B'$ is the strength line.

Proof

$$\text{Taking Figure 4, } \tan W\hat{O}S = \frac{WS}{OS} = \frac{WV+VS}{OS} = \frac{VT+VS}{OS}$$

$$\text{So, } \tan W\hat{O}S = \frac{VS}{OS} + \frac{\sqrt{VR.VS}}{OS} = \tan\phi_1 + \sqrt{\frac{VS^2+VS.SQ}{OS^2}}$$

$$= \tan\phi_1 + \sqrt{\tan^2\phi_1 + \tan\phi_1\cot\phi_3}$$

$$= \tan\phi_1\left\{1+\sqrt{1+\frac{1}{\tan\phi_1\tan\phi_3}}\right\}$$

This is the same as the equation for $\tan\theta$ in case III in para. 3.6. Based on this it is found in Figure 5 that for any value of σ_3 represented by OA, AW' will be the plane of failure.

Also in Figure 5,

$$CA = AA' \tan CA'A = AA' \tan PAC$$
$$= \tau_3 \tan\theta = \tau_3 \sqrt{\frac{\tau_1}{\tau_3}} = \sqrt{\tau_1 \tau_3}$$
$$= \frac{\sigma_1 - \sigma_3}{2}.$$

Hence, C is the centre for the critical circle.

The above procedure can be adopted for case IV of para. 3.6 with slight modification such as the shifting of the origin along the X-axis through the required amount, *viz.*, $K_1 \cot\phi_1, = K_3 \cot\phi_3$ — See also Fig. 2(d).

4.3. **Purely Cohesive Material**

For case II, namely for the purely cohesive material, any circle with centre on the X-axis and radius $= \sqrt{K_1 K_3}$ would represent a critical circle and the slope of the failure plane is given straightaway by $\sqrt{\frac{K_1}{K_3}}$.

4.4. **Material having Parameters on only the Major Principal Plane**

For case XII any ordinate or abscissa, as the case may be, will give the failure plane and the foot of the line, the point circle of failure.

5. DISCUSSION

5.1. From the foregoing sections, it can be seen that given five quantities out of eight involved in three equations, the three other unknowns namely σ_1, τ_1 and τ_3, could be found out both mathematically and graphically. Possibly, because of the original equations being of the form $\sigma_1 - \sigma_3 = 2\sqrt{\tau_1 \tau_3}$, only a non-Euclidean method of graphical solution was given by Barber[3].

It may be seen that by this procedure, both the possible critical circles at failure for a given value of the minor principal stress, could be drawn. Referring to Fig. 3, it can be seen that

the circle with centre C_1 represents the conditions when the maximum shear strength is developed across the plane of maximum principal stress, and the circle with centre C_2, represents the condition when the maximum shear strength is developed across the plane of minimum principal stress.

5.2. This obviously suggests a requisitioning of the assumption (d) in para. 2.1.

In a material with induced anisotropy, in as much as anisotropy is a direct result of the applied stresses or strains, it may be justifiable to assume that the principal strengths develop on the principal planes and also to assume that the maximum shear strength develops on the plane of major principal stress. But this assumption may not hold good for materials possessing anisotropy inherently, unless the planes of principal strengths for the material coincide with the planes of principal stresses. If the two sets of planes do not coincide, but if the orientation between the two sets of planes is known, a procedure similar to the one in the foregoing sections could be adopted with some modifications such as rotating the axes through the required amount etc. However, the latter part of the assumption, namely that the major shear strength is assumed to act on the plane of major principal stress—though justifiable for induced anisotropic materials—may not hold good completely for an inherently anisotropic material, especially when the orientation of the mutually perpendicular planes of anisotropy to the principal planes is very large. Again, for a material which possesses an inherent anisotropy and for which anisotropy is further induced during testing, the assumption involved may have to be modified to a further extent. A sample of an asphalt paving mixture subjected to a non-uniform compaction is very often likely to fall in this category when tested in the laboratory and for such cases, the problem is indeed more difficult to analyse.

5.3. It is interesting to observe that for a purely friction material possessing no cohesion but a varying value of the angle of internal friction, a linear relationship between the principal stresses exists. Further, for such a material, the slope of the failure plane is also a constant showing that irrespective of the values of the applied stresses failure will take place across a plane of given orientation.

A similar result is also observed in a material possessing both variable cohesion and a variable angle of internal friction within the mass for the relation $K_1 \tan\phi_3 = K_3 \tan\phi_1$. Also, the slope of the failure plane is a constant irrespective of the applied stresses. And if the Mohr's system of analysis together with the other assumptions involved could be extended for the tension side also (the negative side on the X-axis), it will be interesting to note, that the point of intersection of the two Coulomb's lines on the X-axis would represent a limiting case of a point circle of failure. This might represent a condition when there is an initial tensile stress equal in all directions within the mass and later on as the external stresses are induced, the strength over both the principal planes increase from a zero value in accordance with the Law of Coulomb, as represented by the two lines. But such an extension may be considered inaccurate for a granular material.

However, although the linear relationship between the principal stresses is different in the two cases mentioned above (cases III and IV in para. 3.6) the slope of the failure plane is the same both for a purely frictional material with values of ϕ_1 and ϕ_3 and no cohesion at all and for a material possessing the same values of ϕ_1 and ϕ_3 but also having a cohesive resistance in accordance with the equation $K_1 \tan \phi_3 = K_3 \tan \phi_1$. This indeed presents an interesting picture.

But, as a result of this an anamoly would arise, when the usual test data for a material are analysed with the help of Mohr's circle of stress. Taking Figure 6, for example, a rupture line PE is drawn tangential to a critical circle with a given value of $OA = \sigma_3$ and $OB = \sigma_1$. According to the usual interpretations, AP will be the plane of failure, OE the inferred cohesion and PEX' will be the angle of internal friction of the material. But then, it follows, based on the argument set forth above that the material under consideration could fall under any one of the following three categories.

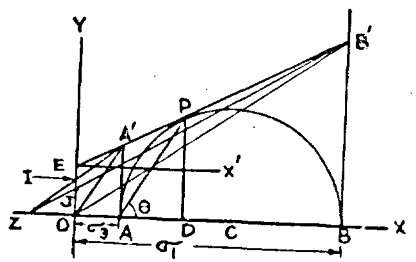


Fig. 6. Mohr's Circle of Stress

(a) Isotropic and possesses both a constant cohesion and a constant angle of internal friction given by OE and PEX',

(b) Anisotropic with no cohesion but possesses a varying angle of internal friction as characterised by angles, $A'OX$ and $B'OX$,

(c) Anisotropic with both varying cohesion as denoted by OI and OJ and a varying angle of internal friction as given by angles IZO and JZO.

The usual assumption being that the material is isotropic it follows that often a material possessing no cohesion at all may be included in the category of cohesion cum friction mass; a material with a varying value of cohesion may be classed as one having constant cohesion and a material possessing different values of the angle of internal friction across various planes may be characterised as one having a single value for the angle of internal friction.

So it could be seen that the conventional method of analysis does not truly reveal the real properties of the material that is tested.

5.4. Further, it can be seen that only in certain specific cases, the relationship between the principal stresses is linear and is of the form $\sigma_1 = \mu\sigma_3 + \lambda$. For the most general case it is of the form

$$\sigma_1^2 + \sigma_3^2 + 2h\,\sigma_1\sigma_3 + f\,\sigma_1 + g\sigma_3 + c = o.$$

which is a general case of a second degree equation in σ_1 and σ_3.

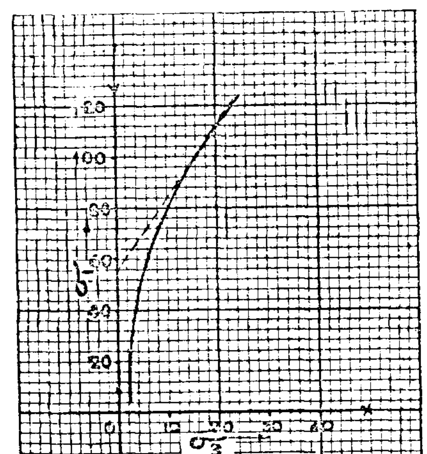


FIG.7. PRINCIPAL STRESS RELATIONSHIP

Also, Figure 7 which is a graph connecting the values of principal stresses at the time of failure for an asphalt mixture clearly shows that a linear law is not always obeyed. However, in such cases it is customary to draw a tangent (as shown by the dotted line in Fig. 7) and to deduce the value of the cohesion and the angle of internal friction for the material. The obvious implication in such a procedure are

(i) that a straight line relationship between the principal stresses at failure is assumed and (ii) that the plane of failure will have a constant slope. But then, it can be seen that the value of cohesion and internal friction so calculated would become doubtful.

5.5. The above disussions clearly reveal that some of the assumptions will have to be revised and their validity ascertained. As far as Mohr's critical circle is concerned—it being a mode of stress analysis—it is well known that it could be employed, irrespective of the fundamental nature of the material that is being studied. It could be taken to be valid for elastic or plastic and for brittle or ductile materials.

It may also be asked as to whether the principles of a force system could be employed for consideration of a strength system as is being done in the method of analysis of shear failure of anisotropic materials. But in as much as the consideration of shear strength is being studied at the time of failure, it may not be far wrong to extend the principles of stress analysis for strength computations at failure as well.

5.6. However, such a doubt will arise, especially when the strength locus on any plane in between the principal planes, is a curve and not a straight line. In fact, this locus could be any curve. But then, as mentioned by Casagrande and Carillo [1], the exact solution will be cumbersome. Also, for small differences between the two angles of internal friction, the assumption of a straight line may be justified.

Finally, it may be said that the anamolies that have crept in, in the present mode of analysis of shear failure are perhaps due to the assumption of validity of Coulomb's Law. There can be no doubt that the simplicity in the present method of analysis is mainly due to this assumption. This assumption may be justified to a certain extent while dealing with completely isotropic materials, but not always for the anisotropic ones. And for the most general case of an anisotropic material, the Coulomb's Law may be taken as valid for shear strengths on individual planes for the sake of simplicity, but surely it cannot be extended further to enable one to consider the envelope of the circles of failure as a straight line obeying the Coulomb's Law.

6. CONCLUSIONS

1. Based on the existing assumptions, general equations for the principal stresses at failure, for the slope of the failure plane etc. for the most general case of an anisotropic material have been worked out.

2. Equations for particular cases of anisotropic materials with different relationships have also been deduced from the general case.

3. A purely graphical method for solving the general equation has also been given. By this method it is possible to draw both the two critical circles of failure for a given condition. It is felt that with this procedure it is easier to get the result without trial and error, than the non-Euclidean method given by Barber[2].

4. A graphical solution for two interesting particular cases has also been suggested, both for obtaining the slope of the failure plane and for the critical circle.

5. A discussion on the various assumptions involved in the studies of shear failure has also been incorporated, from which it follows that the meaning of cohesion and internal friction will continue to be loose, until Coulomb's Law is modified and verified.

6. The present mode of analysis of shear failure does not truly reveal the nature of the material that is being studied and so the need for consideration of anisotropy in the hitherto assumed isotropic materials is all the more greater.

Although the Paper is theoretical—as the practical application of these ideas is still difficult—the Author feels that it might help in clearer thinking and in giving the proper weightage to the test results while interpreting the data.

ACKNOWLEDGMENT

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Paper No. 175

CEMENT CONCRETE SURFACING AND EXPERIMENTAL CONCRETE ROAD CONSTRUCTION ON AGRA—BOMBAY ROAD

By

R. J. DHUMAL, B. E., A. M. I. E. (IND).

AND

M. A. MEHTA, B. E., A. M. I. E. (IND.), Assoc. M. Am. Soc. C. E.

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SYNOPSIS

Beginning with a brief historical introduction, the Paper describes the construction of 23 miles of concrete pavement on the Agra-Bombay Road. The pavement was initially laid in one course with blue trap as aggregate, but rising costs necessitated the adoption of two-course construction with cheaper sand stone aggregate for the bottom. Owing to the scarcity of water in certain reaches membrane curing was carried out.

Taking advantage of the large scale concrete road construction, eleven experimental stretches had been constructed. They were :

1. Unreinforced bonded concrete pavements ;
2. Cement macadam sandwich method ;
3. Cement macadam grouted method ;
4. Slabs of 100 ft and over without joints ;
5. Pavements with varying construction and expansion joint spacings.

The behaviour of these roads under traffic is reported.

The savings in materials and labour by the adoption of the sandwich and grouted methods are brought out.

1. INTRODUCTION

1.1. Brief History of the Road

The road from Agra to Bombay via Gwalior, Shivpuri, Indore, Mhow, Dhulia, Nasik, and the Thul Ghat has a history of a little over a hundred years. It was in 1840 that work was first commenced on the 754 mile long trunk road as part of a large and systematic construction of commercial and military roads throughout the country—a scheme which was soon relegated to the background by the subsequent advance of railway construction. We have on good authority that an annual expenditure of £ 10,000 was at that time authorised on this work[1]*.

The following is a description by Lt.-Col. Goodwyn, Superintending Engineer, Central Provinces, of what the road looked like in the year 1851 :

> " The road for its entire length (excepting the two miles of metal close to Agra) is a track bounded by two parallel lines, six inches wide and deep, twentyone feet apart marking its boundary. It is not raised anywhere except at the approaches to bridges, and it is well that it is not, for as long as it is unmetalled, the natural surface on the whole, excepting the sand,...is preferable to any raising. The sole repairs that it receives, or (in its present state) needs, is simply the filling in of ruts and cuttings after each rainy season, and gradual improvement of the gradients of approach to some of the rivers and nullahs that have not works over them."[2]

* Refers to the item number in the list of references at page 219.

It may be of historical interest to reproduce here the traffic returns annexed by Lt.-Col. Goodwyn to his Report[2]. The branch road referred to is the one that leads from Gwalior to Shivpuri and Kolarus eastwards of the main road. Goodwyn remarks: " The returns of the traffic show an increasing demand for a better road, and this traffic would be greater, doubtless, if the carts could traverse the surface with greater facility."

Abstract of the Traffic Reports for the main and branch road for the months of March, April, May, June, July, August, September, October and November 1850.

Agra and Bombay Road — Agra and Indore Division

Name of Road	Men, women and children	Hackneys	Bullocks	Horses	Camels	Goats, sheep, cows and buffalos	Asses	Sepoys	Elephants	European travellers
Main Road	77,312	3,242	16,728	7,378	414	4,679	793	27	1	6
Branch Road	148,355	5,291	44,490	18,261	1,361	7,563	423	1,487	20	6

In the century between the year 1851 and the year 1954, there has been a progressive improvement in its condition and at the close of the present Five-Year Plan, it is expected that this National Highway will have a modern surfacing throughout its length.

1.2. Necessity of Improvements

The improvements to the road described in this Paper were carried out because of the exigencies of the last World War, and pertain to the length which lies in the former Gwalior State.

Always an important strategic line of communications, the outbreak of the Second World War saw an enormous intensification of civil and military traffic on this Highway. To add to the military traffic there was a drought in the northern parts of the State, necessitating heavy transport of grain and fodder supplies from the southern districts. Also, cessation of coal supplies resulted in a heavy local traffic in firewood and charcoal. Whilst military traffic was mainly convoys of motor trucks, grain, fodder and firewood were largely moved in bullock carts. In 1942, the traffic near Gwalior was 300 to 400 army trucks and 1,500 bullock carts per day.

The increase in traffic, its mixed character, and the fact that the crust of the road was water-bound macadam only 10 ft wide led to a rapid deterioration of the road. Adequate improvements to the road had become a pressing necessity.

1.3. Scheme for Improvement

The sanctioned scheme for improvement provided for the concreting of 25 miles and the surface dressing (2 coats) of another $22\frac{1}{2}$ miles at a total cost of Rs 16 lakhs. Work was commenced in 1944 and continued up to 1949.

1.4. Experimental Lengths

Realising the importance of experimental road construction, short experimental stretches of concrete road employing different specifications and techniques of construction aggregating to about half a mile in mile 84 were planned. Most of these were carried out in the year 1945. Their present condition is very satisfactory and promising of many more years of service.

2. CONSTRUCTION OF CEMENT CONCRETE PAVEMENT

2.1. Organization

A separate construction division was formed and works on the several sections of the road were let out to different contractors on the piece-work system. The Department supplied transport, tools and plant, water vans, pumped water, and cement to the contractors.

The transport consisted of ten lorries which were newly purchased by the Department and rented to the contractors. All repairs to the lorries were carried out at the cost of contractors at a central workshop established and run by the Department.

2.2. Base

The base on which the concrete pavement was laid was an old and very well-consolidated water-bound macadam crust of a thickness generally exceeding 12 inches. The macadam was

normally maintained to a width of 10 ft but it often extended about a foot on either side. The condition of the macadam surface at the time of laying the concrete was rough and uneven. No attempt was made to level this up except that the road was swept and brushed clean of all loose material before the concrete was laid. No insulation of sand or waterproof paper was provided.

The concrete pavement was generally 10 or 12 ft wide. In places where the pavement was wider than 12 ft, concrete of 1 : 4 : 8 mix was laid alongside the edge of the macadam to a depth of 4 in. to provide the necessary base. The expedient proved quite satisfactory.

2.3. Specifications of Concrete Pavement

A total length of 23 miles of concrete pavement was laid, as against the 25 originally sanctioned. Of this, about 8 miles were 10 ft wide and about 15 miles 12 ft. The miles concreted were :

10-ft width

57, 68* to 70, 81 to 84**.

12-ft width

60 to 67, 85 to 91.

The concrete pavement was laid to a camber of 1 in 60 and a uniform thickness of 4 in., although the edge thickness tended to be more due to the existing macadam surface having a steeper camber than that of the concrete slab. The mix used was 1 : 2 : 4. The coarse aggregate was blue trap in two sizes, graded $1\frac{1}{2}$ to 1 in. and $\frac{3}{4}$ in. down, blended in the ratio of 3 : 2 respectively.

Change in Specifications necessitated by Rising Costs. From the commencement of the work there was a continuous rise in the rates of material and labour. Therefore, in order to keep the total cost of the project within the sanctioned amount, two-course construction was adopted, the thickness of each of the courses being 2 inches. Sandstone with a lead of 2 to 6 miles was used for the lower course and blue trap metal, see para. 3.7, from a lead of 8 to 15 miles was used in the top course. The total thickness of pavement being small, the lower course was not separately tamped but only roughly levelled.

Details of the one and two-course construction, costs, the method of curing used, etc., are given in Appendix 1.

* A portion of mile 68 was laid to a width of 12 ft.

** Mile 84 included the experimental lengths.

2.4. Details of Construction

The forms having been laid carefully to line and grade, the macadam base was brushed clean of all loose material and soaked wet (but no pools of water) to prevent absorption of water from the concrete. The concrete was mixed largely in mechanical mixers. The slabs were laid in alternate bays 33 ft long, the ends of the slabs being painted with bitumen before the intermediate bays were filled in. Every third joint, however, was an expansion joint laid with a pre-moulded bituminous filler. On steep grades in the ghat portions, the bays were laid in lengths of 15 ft only. Tamping was carried out by means of the ordinary wooden hand-tamper fitted with a metal shoe. The surface was finished with a belt or a brush. Curing was by ponding for a period of 21 days, except on certain stretches where a proprietary brand of membrane curing compound was used. Water vans with sprinkler attachments were employed to supply water for curing.

The pavement was laid to transition curves wherever necessary and superelevation and widening of pavements to suit were provided in every case. Stone kerbings alongside the edges were provided in the first two miles constructed. In subsequent construction, a 3-ft wide metal shoulder on either side of the pavement was provided instead and it proved extremely useful.

2.5. Curing by Membrane

In arid and semi-arid regions where there is seasonal scarcity of water, membrane curing presents an economic advantage over curing by water and can be equally effective. A continuous seal over the concrete surface is obtained by spraying the curing compound which forms a firm impervious film and prevents the moisture in the slab from escaping by evaporation. The excess water in the concrete—incidental to mixing—effects proper curing.

As an experiment, curing by membrane coating was carried out in miles 60 to 63, 66, 67, 89 and 91.* The curing compound was sprayed on by means of a nozzle immediately the concrete surface was finished.

When the pavement was opened to traffic after the treatment, it was found that iron-tyred traffic caused a very slight distortion of the surface skin. The surface was so well hardened, however,

*These were laid in the summer months, when the temperature was as high as 116 deg. F.

that the distortion has remained intact after 8 years of continuous service. Photos 1 and 2 show the membrane-cured surface of the concrete road in miles 60 and 89.

A gallon of the curing liquid cost at the time Rs 9 and was found to cover 200 sq. ft. of surface. The cost of spraying was annas four per 100 sq. ft. The total cost of curing by membrane was, therefore, Rs 4/12/- per 100 sq. ft. This worked out cheaper than curing by water when the lead for water exceeded two miles.

2.6. Present Condition of Pavement

A recent close inspection of the road revealed the remarkably good condition of the surface after 8 years of heavy duty service with practically no maintenance, except minor repairs in two places. Photos 3 to 6 illustrate the condition of the one and two course road surfaces in miles 82, 83, 89 and 91.

The present traffic intensity on this section is of the order of 800 tons.

Deterioration in Mile 57. In mile 57 a few slabs show surface deterioration in patches which are worn to a depth of about half an inch, Photo 7. The fault is ascribed to an insufficient realization of the need for extra care in initial curing of concrete which was laid in the hot summer months.

Patching in Mile 68/2. A couple of cases of surface deterioration in mile 68/2 are interesting in the method employed to patch them. The surface of the patch was cut around neatly to a depth of 1 inch and was thoroughly cleaned of all loose material by means of a wire brush and wetted. A powder admixture supplied by a Hyderabad firm was mixed dry with portland cement and then prepared into a stiff paste with sand and water. The paste was applied to the patch, tamped and trowel finished. The repair was carried out in 1946 and a recent inspection showed that it had merged perfectly with the adjoining surface and was in excellent condition, Photo 8. In the repair of another patch of size 2 ft by 2 ft, also shown in Photo 8, at the right hand bottom corner, the powder admixture was not used but the patch was painted with a neat cement grout before being filled in.

3. SPECIFICATIONS FOR THE EXPERIMENTS

3.1. In mile 84, twenty experiments were scheduled. But only 11 experimental lengths were constructed. Appendix 2 gives the brief particulars of the experimental lengths with the specification adopted in each case, date of laying, costs, and remarks of inspection on two different dates—half a year and $6\frac{1}{2}$ years after completion of the work.

The present traffic intensity in this mile is 610 tons per day, of which 45 per cent is iron-tyred traffic. In the harvest season, the traffic might be more.

The various experiments which were carried out can be grouped as follows:

(1) Unreinforced bonded concrete (Experiments No. 1 & 2)

(2) Cement macadam (sandwich) (Experiments No. 3/A & B)

(3) Cement macadam (grouted) (Experiments No. 4/A & B)

(4) Slabs 100 ft and over in length, without joints (Experiments No. 12/A, B & C)

(5) Pavements with varying contraction and expansion joint spacings (Experiments No. 14/A & B)

Fig. 1, is a chart showing the layout of these experiments. The purpose of each of the experiments, the method of construction, comments of the Authors, and other relevant information are given later in Sections 4 to 8.

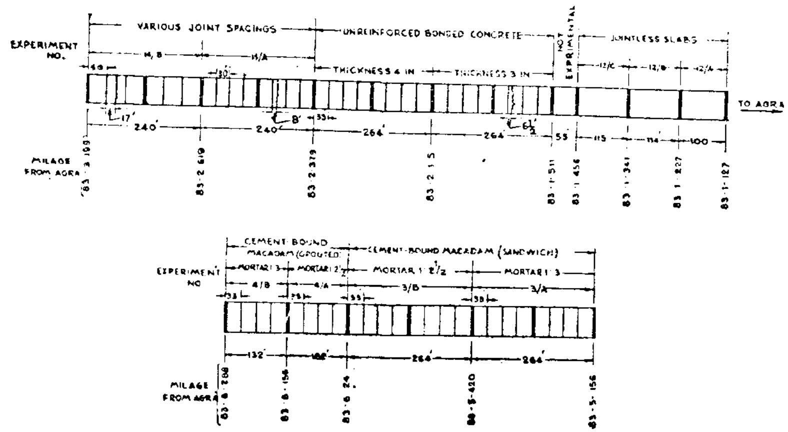

FIG. 1

3.2. Organization

Because of the necessity for exercising close control, the experimental work was carried out departmentally by the con-

struction division of the Gwalior P.W.D. with the co-operation of the Concrete Association of India.

Labour was recruited from peasant population of the area and some of them had experience of concreting on the construction of the Maharajpur runway (near Gwalior).

3.3. Costs

Quantities of materials consumed and costs were accurately kept. The experimental lengths being very short involved constant change of method and materials. With longer lengths, the unit costs should work out less because of the scope afforded for lower overheads.

3.4. Base

The base on which the experimental pavements were, laid was identical to that described in para. 2.2, viz., old and very well consolidated water-bound macadam crust of a thickness generally exceeding 12 in. and of width about 12 feet

The slabs of all the experiments were keyed to the macadam surface which was thoroughly wire brushed and wetted prior to laying concrete. No insulation of sand or waterproof paper was provided in any case.

3.5. Width and Thickness of Pavement

The experimental pavements were all 10 ft wide with 3 ft wide metal shoulders which were provided later on either side. The thickness was specified variously as 3, 4, $4\frac{1}{2}$ and 5 inches. These thicknesses were measured along the centre line of the pavement (or at one edge on superelevated lengths) but, owing to the steeper camber of the base crust, edge thicknesses were greater by $\frac{1}{2}$ to $1\frac{1}{2}$ inches. No attempt was made to re-grade the base by a lean mix or otherwise. The surface camber was 1 in 60.

3.6. General Specifications

All the experiments employing premixed concrete, viz., 1, 2, 12 and 14, were one-course pavements constructed in accordance with the specification in para. 2.3, with the following modifications:

(1) Coarse aggregate used was in two sizes graded $1\frac{1}{2}$ to 1 in. and $\frac{3}{4}$ to $\frac{3}{8}$ in. and mixed in the ratio of 50 : 50 and later 70 of the larger size and 30 of the smaller size.

(2) Unless otherwise stated, dummy joints were introduced at 33 ft intervals and $\frac{3}{8}$ in. thick expansion joints at 132 ft.

3.7. Rates of Material

The following rates of materials obtained at site:

Blue trap ($1\frac{1}{2}$ to $\frac{3}{4}$ in.)	Rs 28 per 100 cu. ft.
($\frac{3}{4}$ to $\frac{3}{8}$ in.)	Rs 65 ,, ,, ,,
Sandstone	Rs 18 ,, ,, ,,
Sand (Simiria, coarser variety)	Rs 40 ,, ,, ,,
Sand (Parbati, finer variety)	Rs 20 ,, ,, ,,
Cement from Banmore Cement Factory 25 miles away	Rs 48 per ton.

3.8. Sieve Analyses of Sands

The following are the sieve analyses of the Simiria and Parbati sands:

Sieve size	Per cent coarser	
	Simiria	Parbati
No. 4	0	0
8	9	4
14	31	8
28	71	68
50	95	91
100	99	98
Fineness Modulus	3.05	2.69

3.9. Curing

Concrete was covered with damp hessian or gunny bags soon after finishing operations. These were removed from 12 to 24 hours later and the pavement cured under wet earth for a period of 21 days, except for experiments No. 12/A and 12/B, where curing was done for 10 and 15 days respectively.

3.10. Plant

A 10-7 and a $5\frac{1}{2}$-$2\frac{1}{2}$ concrete mixers were made available. For the cement macadam (sandwich) experiment a three-wheeled 6-ton roller was used.

Photo 1. **Surface of membrane cured concrete road in Mile 60**

Photo 2. **Surface of membrane cured concrete road in Mile 89**

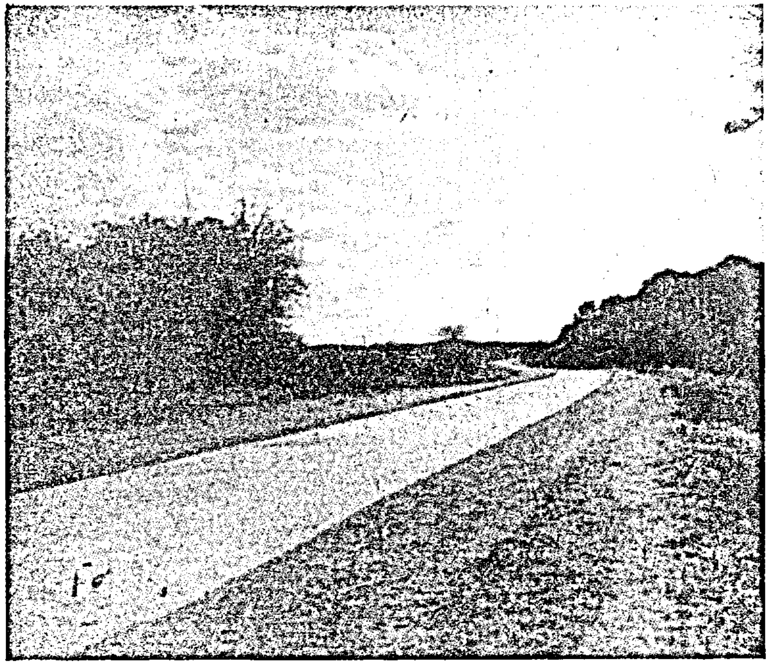

Photo 3. **One-course concrete road in Mile 82**

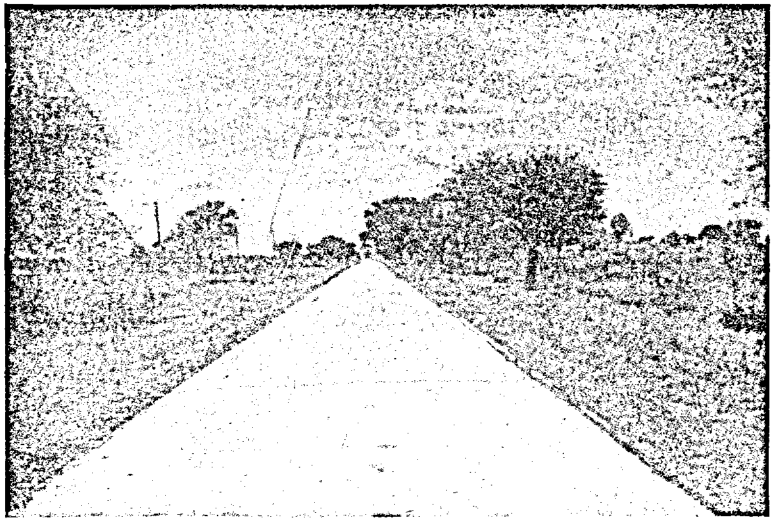

Photo 4. **One-course concrete road in Mile 83**

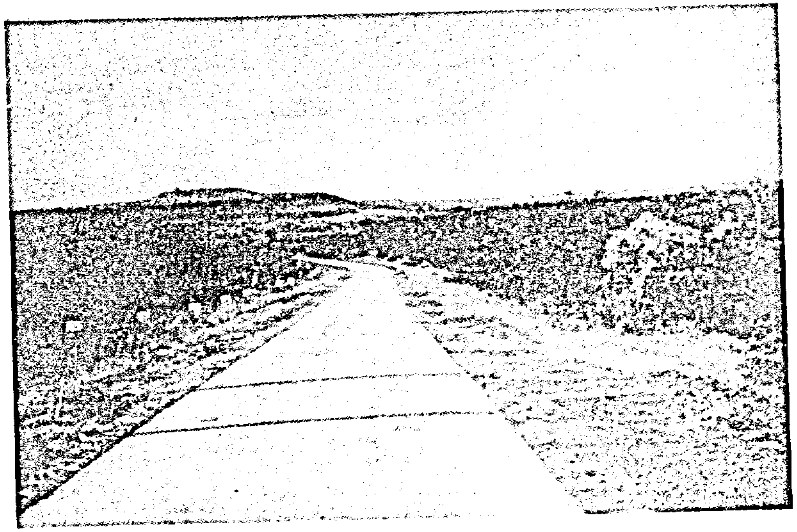

Photo 5. **Two-course concrete road in Mile 89**

92851

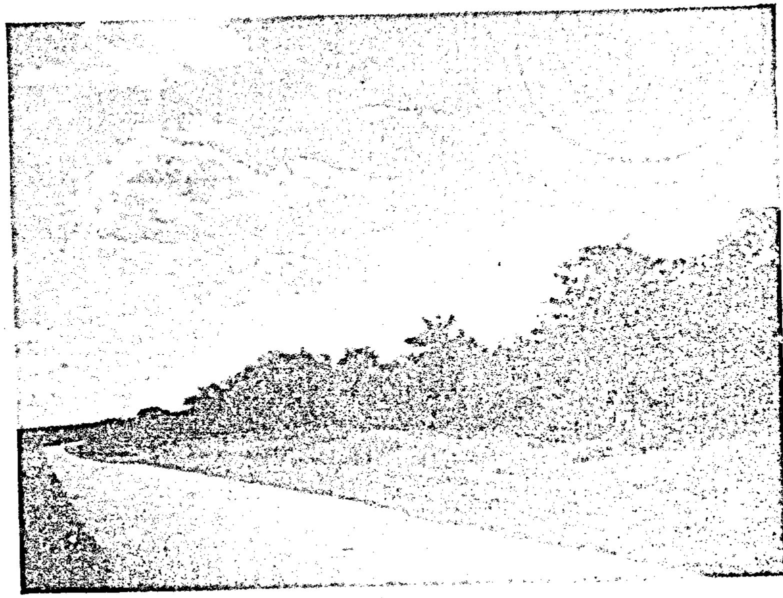

Photo 6. **Two-course concrete road in Mile 91**

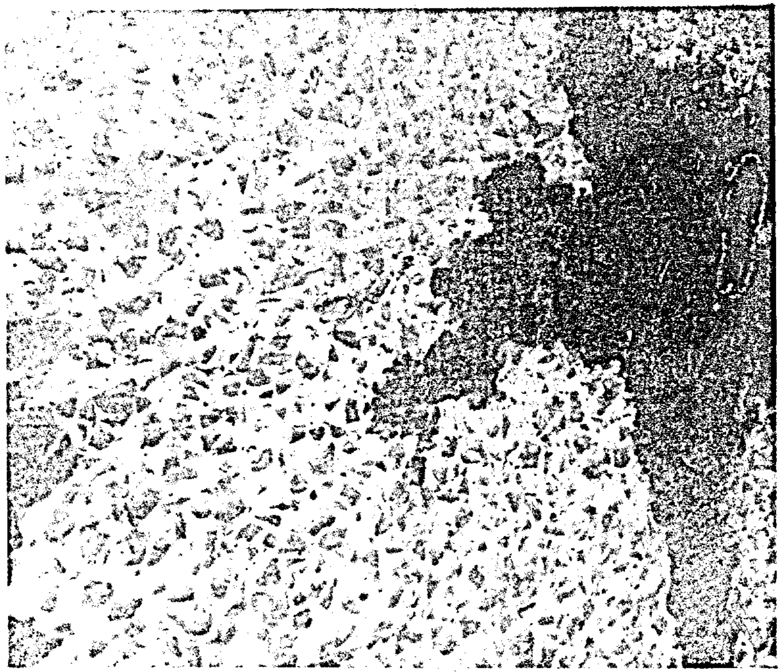

Photo 7. **Surface deterioration in patches of a slab in Mile 57**

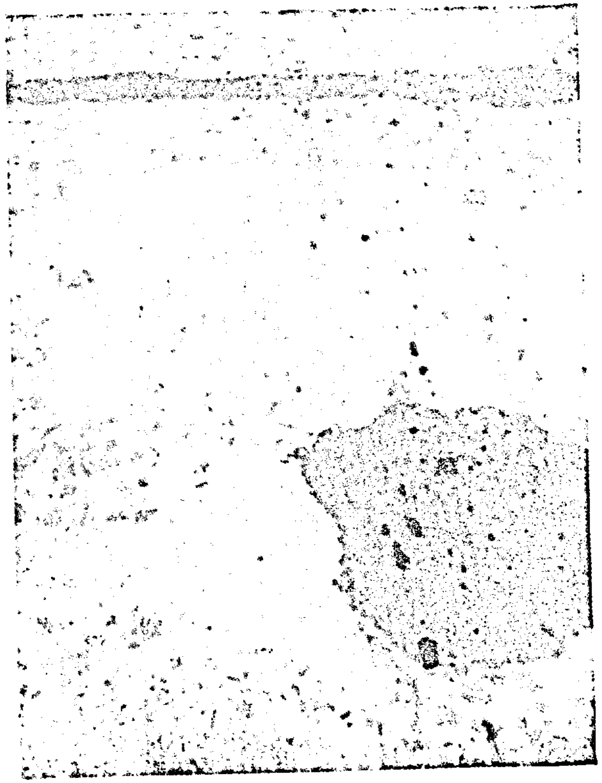

Photo 8. **Patches in Mile 68/2**

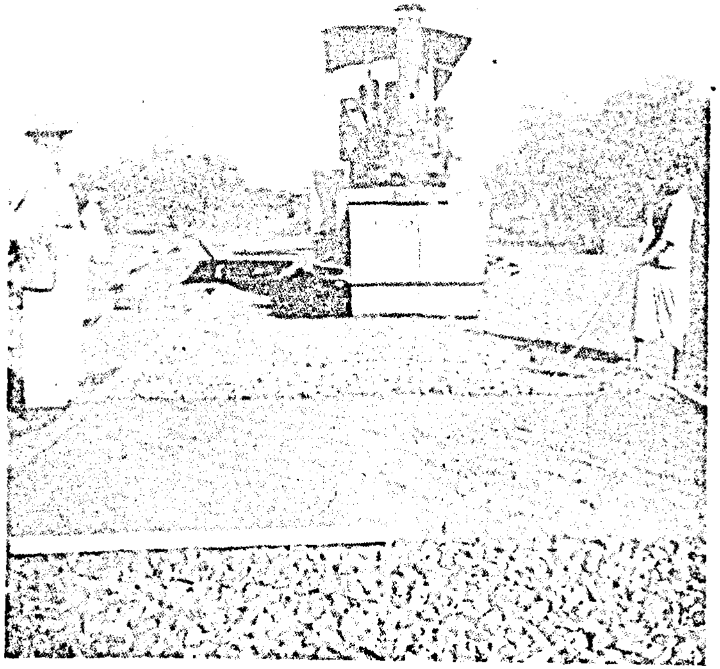

Photo 9. **Cement-Bound Macadam in Experiments 3/A & 3/B**

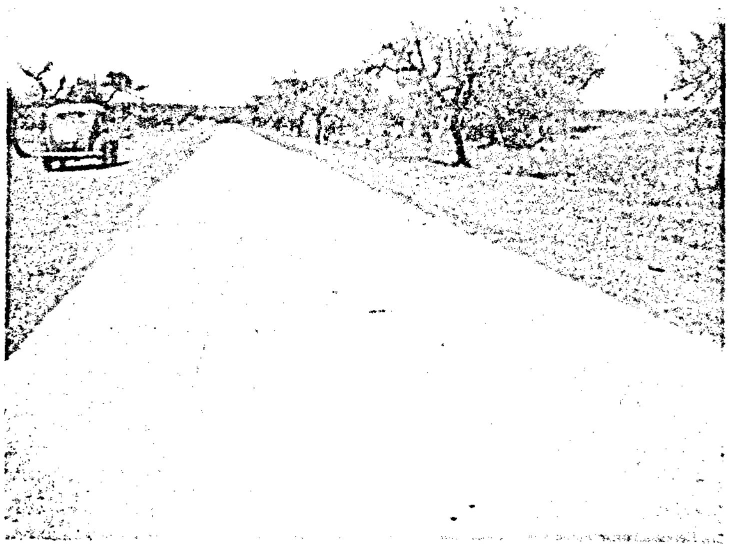

Photo 10. **A view taken on 4-4-52 showing the excellent condition of Experiment No. 3/A (Cement macadam sandwich with premixed top) after 6½ years of service**

Photo 11. **A close-up showing the uniformly worn surface of Experiment No. 3/B**

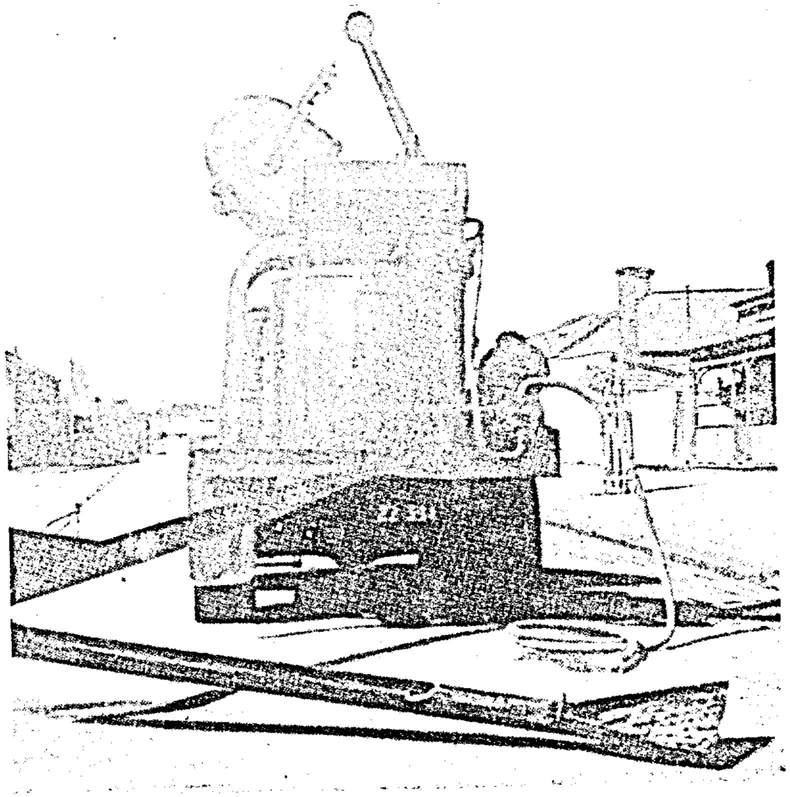

Photo 12. **Grout mixer**

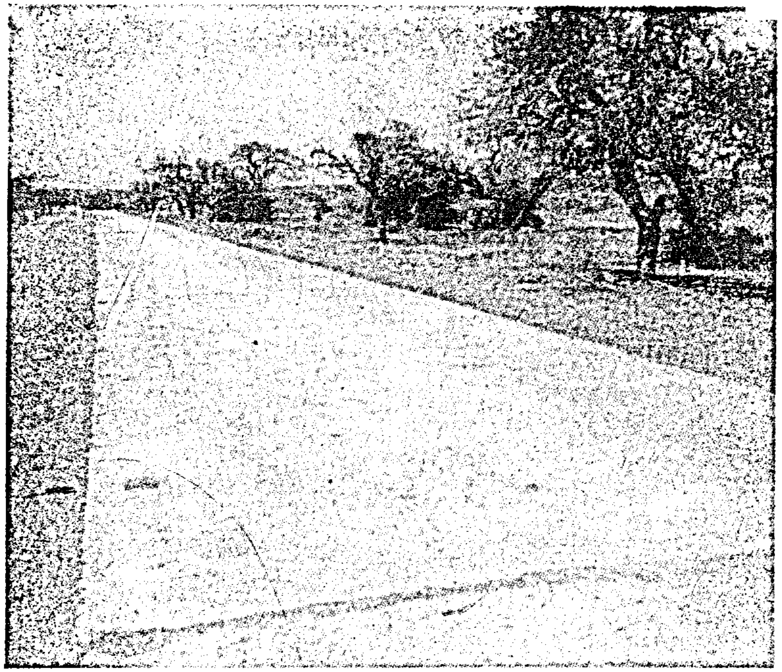

Photo 13. **A view showing the excellent condition of Experiment No. 4/A**

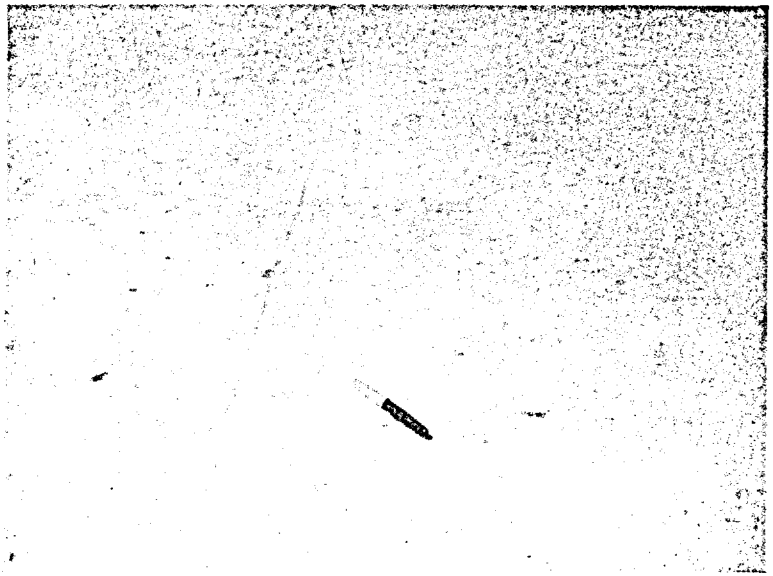

Photo 14. **A close-up showing the uniformly worn surface of Experiment No. 4/A**

4. SLABS OF 100 FEET AND OVER IN LENGTH WITHOUT JOINTS

(Experiments 12/A, B and C)

4.1. Object of the Experiment

The experiment was devised to find out the minimum length which remained uncracked in a long slab initially constructed without joints. This would give some information with regard to the maximum spacing of contraction joints when expansion joints are placed at 100 ft or so intervals.

Three slabs of lengths 100, 114, and 115 ft were laid and cured for 10, 15 and 21 days respectively. Curing periods were varied in order to watch any possible effect of duration of curing upon the length of uncracked slab.

4.2. Behaviour of the Slabs

Between five and six weeks after laying, one transverse crack each over the full width of the pavement was found in 12/B and 12/C extending. In 12/B, the crack divided up the 114 ft long pavement into slabs 48 and 66 ft long. In 12/C the 115 ft long pavement was divided into slabs of 80 and 35 ft long. No crack was visible in 12/A up to the 25th of November 1945, almost two months after laying the concrete.

In May 1946, eight months after laying, there was no change in 12/B, but in 12/C the 80 ft slab portion had divided further into 53-ft and 27-ft lengths. The 100-ft long slab, 12/A had cracked into lengths of 58 and 42 ft.

There were no further cracks till to-day. Fig. 2, shows diagrammatically the cracks. Whilst it is difficult to draw conclusions from these limited trials, it is, nevertheless, interesting to note that the minimum uncracked slab lengths in 12/A and 12/B are considerably longer than the joint spacings normally allowed.

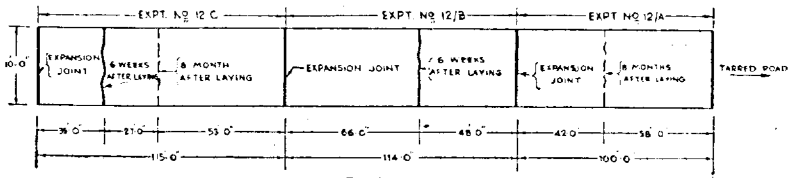

FIG. 2

5. VARYING CONTRACTION AND EXPANSION JOINT SPACINGS

(Experiments 14/A & B)

5.1. Object

The experiments were intended to test the effects of various combinations of spacing of dummy-groove contraction and expansion joints.

The general practice in India as regards joints at that time was to space contraction joints at 30 to 33 ft intervals and to provide $\frac{3}{8}$ in. thick expansion joints at 90 to 100 ft intervals. The contraction joints were 'plain butt' type and concrete was laid by the 'alternate-bay' method.

5.2. The Dummy-groove Contraction Joint

The dummy-groove (Fig. 3) has certain advantages over the plain butt type of contraction joint. It permits continous construction and provides excellent anchorage for the joint sealing material as the groove extends to only about one-quarter to one-third of the thickness of the pavement. When the concrete shrinks, the tensile stresses set up will cause the slab below the groove to crack. There still remains, however, a certain amount of aggregate interlock below the groove which is useful for the transfer of loads across the joint.

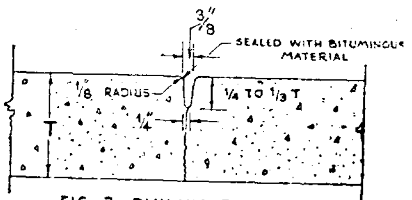

FIG-3 DUMMY JOINT

These considerations led to the adoption of the dummy-groove contraction joint for all the experiments at Gwalior.

5.3. Expansion Joints at Long Distances an Advantage

A concrete pavement built, when the prevailing temperature is around 70 deg. F., will expand $50 \times 0.0000055 = 0.000275$ in. for every inch length in the summer months when the temperature rises to 120 deg. F. If this expansion is not allowed to occur and if the modulus of elasticity of the concrete is 3,000,000 lb per sq. inch the compressive stress which will develop will be $0.000275 \times 3{,}000{,}000 = 825$ lb per sq. inch. If this stress did develop it would be of little significance to the concrete pavement which has, generally, a compressive strength of 4,000 lb per sq. inch, but under good maintenance conditions it will not develop. For, during the hardening of the concrete and subsequent drying the pavement shrinks, opening contraction joints and leaving space for a considerable amount of expansion before restraint is, encountered. It is, therefore, important to exclude inert incompressible matter from entering the contraction joints (or cracks), and if this is done, the compressive stress developed due to expansion would be much less than 825 lb per sq. inch.

In fact, the compressive stress which does develop is not harmful but is beneficial to the pavement. When the pavement is in compression the contraction joints are tightly closed. At temperatures below that obtaining at the time of placement, the contraction joint opening will be at a minimum, thus assuring satisfactory load transfer. Again, to the extent that compressive stresses are present, the pavement is in a state of prestress which tends to reduce tensile stresses in the top or bottom of the slab due to flexure under loads.

Experience in America had shown that it was a distinct advantage to have very little or no expansion space in concrete pavement if dummy-groove contraction joints were properly maintained to exclude foreign inert material.

5.4. The full programme in this group of experiments was as follows:

(a) Dummy-groove joints at 30 ft and $\frac{3}{4}$ in. thick expansion joints at 210 ft, No. 13.

(b) Dummy-groove joints at 30 ft and $\frac{3}{4}$ in. thick expansion joints at 120 ft, No. 14/A.

(c) Dummy-groove joints at 40 ft and $\frac{3}{4}$ in. thick expansion joints at 120 ft, No. 14/B.

(d) Dummy-groove joints at 30 ft and $\frac{3}{4}$ in. thick expansion joints at 300 ft, No. 15.

The above considerations led to the planning of the above experiments where the expansion joint spacings were greater than the usual. Unfortunately, the experiments with the longer spacings (210 and 300 ft) were not carried out. The length of experiments 14/A and B was, however, increased to 240 ft each, Fig. 1. The Authors recommend to engineers to carry out tests of unreinforced pavements with expansion joints spaced at 400 and 600 ft whenever they have an opportunity of doing so.

5.5. Behaviour of the Slabs

(a) **Experimental Section 14/A.** Of the eight slabs of 30 ft length each, only one (the sixth) developed a transverse crack 8 ft away from the dummy joint.

(b) **Experimental Section 14/B.** Of the six slabs of 40 ft length each, only one (the second) developed a transverse crack, 17 ft away from the dummy joint.

(c) Considering that the contraction joint spacing of 30 and 40 ft is much in excess of that recommended for crack control, it is noteworthy that there were not many transverse cracks in this experiment.

(d) A total of 54 slabs of lengths 30, 33 and 40 ft and aggregating 1,800 ft was laid, of which only three developed transverse cracks, one each (See Fig 1). Details are as follows:

Experiment No.	Spacing between contraction joints	Total No. of slabs	Slabs showing transverse cracks
14/A	30 ft	8	1
1,2,3/A&B, 4/A & B	33 ft	40	1*
14/B	40 ft	6	1

This type of contraction joint may therefore be considered as complete success. It has since been adopted on the construction of the important heavy-duty Messent Road at Bombay[4], and has now come into fair use in India.

6. UNREINFORCED BONDED CONCRETE

(Experiments 1 & 2)

6.1. Object

These experiments were devised to find out the minimum thickness of unreinforced concrete slab keyed to a well-consolidated existing macadam surface that can stand the normal traffic requirements.

* The spacing here is in excess of that normally used for control of cracking with the type of aggregate used.

Experimental sections 1 and 2 are each 264 ft long. The centre thickness of the slab in No. 1 is 3 in. and that in No. 2 is 4 inches. The slabs were laid to the usual specification except that particular care was taken to scrub the macadam base free of dust and all loose material so as to get the concrete properly bonded to it.

6.2. Behaviour

Both the experimental lengths are today in a sound condition except that there is a transverse crack in the sixth slab of the Experiment No. 1, 6½ ft from the dummy-groove joint, see Fig. 1. The surfaces of both the experiments are in an excellent condition. The experiments may be regarded as an unqualified success and as confirming the experience gained elsewhere that a 3-in. thickness of concrete can take fairly heavy traffic where there is an existing well consolidated crust of water-bound macadam and the concrete is so laid as to bond thoroughly with the base. The economy of this specification hardly needs to be mentioned.

7. CEMENT-BOUND MACADAM (SANDWICH METHOD)

(Experiments 3/A and B)

7.1. Object of the Experiments

The cement bound macadam experiments employing the sandwich and the grouted methods of construction, were put in to see if the initial cost of construction of the conventional premixed concrete road could not be lowered, without affecting the structural strength and the riding quality.

The conventional premixed concrete pavement is uniform in strength throughout the depth of the pavement. Abrasion from the traffic is taken solely by the surface. Therefore, whilst the crushing strength and abrasive resistance of the top layer of a concrete pavement have to be high, there is scope for a comparatively weaker lower layer, particularly when the pavement is laid on an existing well consolidated water-bound macadam which serves as an excellent foundation.

This is the concept on which the specifications for the cement-bound macadam experiments were based. The top 1½ in. was premixed concrete 1 : 2 : 3½ with tough, durable trap aggregate. Below this was a 3½ in. consolidated thickness of sandstone, an aggregate softer and cheaper than trap, bound together with a 1 : 2½ or 1 : 3 mortar grout.

7.2. Construction by the Sandwich Method

A layer of 2 to $\frac{3}{4}$ in. graded sandstone about 2 in. deep was spread on the prepared macadam base. The next day the stone was lightly rolled to shape (but not to consolidation) with a three-wheeled 6-ton roller. This was, however, discontinued as an appreciable proportion of the stone was seen to crumble under the roller.

Cement mortar using Simiria sand, see para. 3.8, and moderately stiff in consistency was then spread to a uniform depth of $1\frac{1}{2}$ in. on the stone which was previously moistened to prevent absorption of water from the mortar. The proportions were 1 volume of portland cement to $2\frac{1}{2}$ volumes of sand for experiment 3/A and 3 volumes of sand in the case of 3/B. Another 2-in. layer of sandstone, again graded from 2 to $\frac{3}{4}$ in., was laid over the mortar. The whole was then consolidated by rolling with a three-wheeled* 6-ton roller, working from the sides of the pavement towards the middle. Steel forms had been used (as for the rest of the experiments) and had been spiked carefully into position to prevent movement due to thrust from the roller.

During rolling, the mortar gets first pressed into the voids in the bottom layer and then it works up to fill the voids in the top layer. When the mortar came up, rolling was stopped, and excess mortar was brushed over to distribute uniformly. A little water may be sprinkled on the wheels of the roller to prevent stones in the surface from picking up.

Photo 9 shows the four principal stages in the construction of the cement bound macadam base which was consolidated to a thickness of $3\frac{1}{2}$ inches.

The top course of $1:2:3\frac{1}{2}$ concrete using maximum size $\frac{3}{4}$ in. hard trap aggregate was laid to a depth of $1\frac{1}{2}$ in. and finished in the usual manner. There was no appreciable time lag between the finishing of the cement bound macadam underlayer and the laying of the top wearing concrete.

7.3. Observations on Construction

A certain amount of sandstone crushed under the roller, but it is considered that the strength of the sandwich layer was not thereby impaired. The day's work limit was set at 132 ft only. Therefore, the roller was idle for long periods. With a larger programme, almost three times the above length, can be completed with ease and considerable economy.

*A tandem roller is preferable for this kind of work but was not available.

The Authors feel that the thickness of the premixed top coat may very well be reduced to one inch. This would further reduce the consumption of cement.

7.4. Behaviour of the Test Length

After more than 8 years of continuous heavy traffic, both the experimental lengths 3/A and 3/B are in sound structural condition and their wearing surfaces are excellent, Photos 10 and 11. Judging by past behaviour, this type of pavement can be expected to last as long as the conventional concrete road. Soon after the experiments were completed, examination of a test piece removed from the edge of the sixth slab in experiment No. 3/A showed a thorough distribution of the cement-sand mortar throughout the thickness of the layer.

7.5. Economy in Materials and Cost

The approximate quantities of material required per 100 cu. ft. of the finished pavement are given in Table 1.

TABLE 1

Quantities of Materials required for Sandwich Construction

Type of pavement	Cement per 100 cu. ft. cwt.	Sand per 100 cu. ft. cu. ft.	Stone per 100 cu. ft. cu. ft.
Cement macadam sandwich			
1 : 2 mortar	$13\frac{1}{4}$	33	110
1 : $2\frac{1}{2}$ mortar	12	$37\frac{1}{2}$	110
1 : 3 mortar	$10\frac{3}{4}$	$40\frac{1}{2}$	110
Premixed concrete			
1 : 2 : $3\frac{1}{2}$	$20\frac{1}{2}$	49	85
1 : 2 : 4	$18\frac{1}{2}$	45	90

Taking a 132 ft length of pavement width 10 ft and thickness 5 in. uniform, the quantities required for specifications used in experiments 3/A and 3/B and for a premixed concrete pavement are given in Table 2.

The saving in cement over normal construction is 21.5 per cent in the case of 3/A and 26.5 per cent in the case of 3/B. Also, there is saving in the cost of stone which, at the rates given in para. 3.7, works out to about 11 per cent. Savings in time and labour due to the greater speed in construction and simpler operation compared to normal construction and the fact that 70 per cent of the coarse aggregate does not have to pass through a mixer contribute to further economy. These can be realised to a larger extent if a big enough construction using this method is undertaken.

TABLE 2

Quantities required for 132 ft length of Sandwich Construction

Type of pavement	Cement cwt.	Sand cu. ft.	Stone cu. ft.
As in experiment 3/A			
Base course	46 } 80	144 } 225	425 sandstone
Top course	34	81	140 trap ($\frac{3}{4}$ in.)
As in experiment 3/B			
Base course	41 } 75	156 } 237	425 sandstone
Top course	34	81	240 trap ($\frac{3}{4}$ in.)
Premixed 1 : 2 : 4 concrete 5 in. thick	102	270	470 trap ($1\frac{1}{2}$ in.)

8. CEMENT-BOUND MACADAM (GROUTED METHOD)

(Experiments 4/A and B)

8.1. Basis of the Specification Employed

The objective in experimenting with the grouted, or cement penetration method as it is sometimes called, was the same as that of the sandwich method, viz., an attempt to lower the initial cost of construction of a concrete road without impairing its quality in service.

It may be mentioned here in passing that the grouted road as constructed at Gwalior in 1945 was first of its kind built in India. Subsequent experiments have been carried out in many parts of India.[5]

8.2. Fluidity of Grout

Many engineers with whom the Authors discussed this type of construction in 1945 expressed doubt about the possibility of complete penetration of the grout to the full depth. It is essential in this type of construction to have the voids in the stone of such series as will allow the grout to flow through them freely. To this end all aggregates below $\frac{3}{4}$ in. size are excluded. Again, the capacity for penetration of grout is determined by its fluidity in relation to aggregate size, that is to say, the grout has to be more fluid to penetrate through smaller size aggregates while it could be thicker if larger size aggregate is used.

The fluidity of grout is influenced by the following three factors :

(1) the proportions of cement and sand in the mix,

(2) the size and grading of sand, and

(3) the amount of mixing water.

It is important to be able to measure accurately the fluidity of grout. This is done by means of a "flow cone" developed by the Portland Cement Association of Chicago, U. S. A. and first used on experimental lengths of grouted pavement carried out at Elmhurst, Illinois, in 1933[6]. In Appendix 3 details of the flow cone and the relation of the fluidity of the grout to the size of aggregate are given.

8.3. Construction by the Grouted Method

Sandstone was spread between the forms on the prepared macadam base and tamped roughly to shape to a depth of $3\frac{1}{2}$ in. by means of a notched tamper. This operation was completed for the full length the day before grouting was done. No rolling was carried out as experience with the sandwich experiment had indicated that the sandstone crushed under the roller. Rolling has the advantage that it compacts the aggregate, reducing voids and thereby grout consumption. The tests at Elmhurst previously referred to had indicated average compaction percentages of different types of coarse aggregate with different methods of compaction and these are given in Table 3.

TABLE 3

Average Compaction Percentages for Different Coarse Aggregates and Methods of Compaction

Initial compaction by	Final compaction by	Crushed limestone Per cent	Gravel Per cent	Crushed slag per cent
5.8 ton roller	5.8 ton roller	14	—	14
None	5.8 ,, ,,	5	6	8
3.3 ton roller	3.3 ,, ,,	6	—	—
None	3.3 ,, ,,	2	2	—
Vibration	Vibration	4	10	6
None	Vibration	5	5	10
None	Hand-tamping	2	—	—

It will be seen that the compaction varies from 2 to 14 per cent, effecting a corrresponding saving in the consumption of grout. Against this must be set off the hire costs of the roller and an additional process in the construction.

Since the stone grading was $\frac{3}{4}$ in. to 2 in., a fluidity of grout of 19–21 seconds was called for, Appendix 3. As the aggregate was of small size and the grout mixes employed rather lean, the finer Parbati sand (a medium sand graded 0-8, see para. 3.8) which gave the required flow time was selected. It was also considered that with the available plant and the method to be employed in grouting, the Parbati sand grout would be easier to handle.

The grout was mixed in a concrete mixer, the amount of water added being so regulated as to give a flow time of 19-20 seconds. Tests with the flow cone were carried out frequently during the progress of the work. The grout was brought over from the mixer in pails, and poured over the stones (which had been wetted previously) until all voids were filled. Excess grout was brushed over the surface uniformly. Holes dug into the stones down to the macadam a couple of feet ahead of the grouted surface showed unsegregated grout flow along the bottom into the hole, indicating good penetration.

The mix proportions for the top premixed concrete were 1 : 2 : 3½. Hard trap aggregate of a maximum size of $\frac{3}{4}$ in. was used, and the concrete was laid to a depth of 1½ in. consolidated thickness and finishned in the usual manner. The water-cement ratio for this concrete was, however, kept very low as after the grouting of the base a certain amount of free water or very thin grout came up to the surface whilst the top course was being tamped and finished.

8.4. Observations on Construction

The normal concrete mixer designed to mix concrete with coarse aggregate is not necessary for mixing grout. Special grout mixers have been devised and used in Australia where a large amount of cement-grouted road work has been carried out. They are generally fitted with a distributing arm which facilitates the grouting operations considerably. Photo 12 is a back view of the mixer, showing 1-in. water hose attached to the water cylinder and distributing arm in position for grouting. The distributing arm has flexible joints. Such plant is not costly and would be useful in any large-scale programme of road building by the grouted method.

The Authors feel that the thickness of the premixed top coat may be reduced to 1 in. This would contribute to economy in the consumption of cement.

A suggested specification for cement-grouted pavements based on the experience gained in the construction of this experiment are given in Appendix 4.

8.5. Behaviour of the Slabs

Except for cracking around the expansion joint between experiments 4/A and 4/B for a distance of 2 to 4 ft on either side, Appendix 2, which is attributed to local defect, both the pavements are in a sound structural condition and the wearing surface is excellent after 8 years of service. Photos 13 and 14 show a close-up and a general view of the experimental lengths.

8.6. Economy of Materials and Cost

The approximate quantities of material required per 100 cu.ft. of finished pavement are given in Table 4.

TABLE 4

Quantities of Materials required for Grouted Construction

Type of pavement	Cement per 100 cu. ft. cwt.	Sand per 100 cu. ft. cu. ft.	Stone per 100 cu.ft. cu. ft.
Cement macadam (grouted)			
1 : 2 grout	14	35	110
1 : 2½ ,,	12¼	38	110
1 : 3 ,,	10½	39½	110
Premixed concrete			
1 : 2 : 3½	20½	49	85
1 : 2 : 4	18½	45	90

The quantities for the grouted cement macadam are calculated as follows :

Assume :

Percentage of voids in the stone	40
Weight of surface dry loose sand	100 lb per cu. ft.
Specific gravity of sand	2.65
Specific gravity of cement	3.15

It is found that 8 gallons of water are required to produce the desired fluidity of 19–20 seconds in the grout. See Plate I.

Then, making use of the concept of absolute volumes[7], one cwt of cement will produce :

$$\frac{112}{3.15 \times 62.4} + \frac{250}{2.65 \times 62.4} + \frac{80}{1 \times 62.4} = 3.36 \text{ cu. ft. of grout.}$$

The cement required to make sufficient grout to fill the 40 cu. ft. voids in the stone is, therefore,

$$\frac{40}{3.36} = 11.9 \text{ cwt.}$$

It has been found by experience, however, that a volume of grout averaging 17 per cent in excess of the volume of voids in the coarse aggregate of the fully grouted pavement is required to compensate for the loss of grout and shrinkage in grout volume due to escaping water.

The cement required is, then,

$$1.17 \times 11.9 = 14 \text{ cwt.}$$

The quantity of sand required is twice the volume of cement and, assuming one cwt of cement measures $1\frac{1}{4}$ cu. ft.

$$\text{Sand} = 1\tfrac{1}{4} \times 14 \times 2$$
$$= 35 \text{ cu. ft.}$$

Allowing for wastage, etc., the quantity of stone required is taken as 110 cu. ft.

Taking a length of pavement of 132 ft, width 10 ft, and 5 in. uniform thickness, the quantities required for pavements with specifications as in experiments 4/A and 4/B and for a premixed concrete pavement are given in Table 5.

TABLE 5

Quantities required for 132 ft length of Grouted Construction

Type of pavement	Cement cwt.	Sand cu. ft.	Stone cu. ft.
As in experiment No. 4/A			
Base course 47 }	81	147 } 228	425 sandstone
Top course 34 }		87 }	140 trap ($\frac{3}{4}$ in.)
As in experiment No. 4/B			
Base course 41 }	75	152 } 223	425 sandstone
Top course 34 }		81 }	140 trap ($\frac{3}{4}$ in.)
Premixed 1 : 2 : 4 Concrete 5 in. thick	102	270	470 trap ($1\frac{1}{2}$ in.)

The saving in cement compared with the conventional constuction is 20.5 per cent for experiment 4/A and 26.5 per cent for 4/B. The savings in the cost of stone and labour are the same as in the sandwich method, see para. 7.5.

9. CONCLUSIONS

9.1. Two-course cement concrete road pavements using a 2-in. bottom course of locally available sandstone and a top course of 2-in. premixed concrete have been a success.

9.2. Pavements cured by membrane curing compounds, in places where there was scarcity of water, have behaved well.

9.3. The experiments yielded valuable data on the spacing of contraction and expansion joints.

9.4. Where the existing well consolidated water-bound macadam base is available, bonded concrete roads 3 to 4 in. thick provide a successful and economical construction.

9.5. Cement-bound macadam $3\frac{1}{2}$ in. thick using sandstone for the bottom course and $1\frac{1}{2}$ in. of premixed concrete 1 : 2 : $3\frac{1}{2}$ for the top has behaved satisfactorily; it offers considerable scope for economy.

ACKNOWLEDGMENTS

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APPENDIX 1

Details of the Concrete Pavement

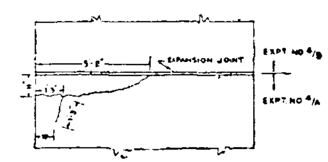
Mile	Width of pavement ft	Specification employed	Cost Rs	Lead in miles				Method of curing
				Sandstone	Blue trap	Sand	Cement	
57	10	One-course	28,550	—	18	43	—	Ponding
60	12	Two-course	37,447	6	15	40	4	Membrane curing
61	12	do.	50,778	5	14	39	3	do.
62	12	do.	32,609	4	13	38	2	do.
63	12	do.	36,226	3	12	37	1	Ponding and membrane curing
64	12	One-course	32,149	2	11	36	1	Ponding
65	12	Two-course	34,737	3	10	35	2	do.
66	12	do.	32,224	4	9	34	3	Membrane curing
67	12	do.	33,133	5	8	33	4	do.
68	10 & 12	One-course	40,490	—	7	32	5	Ponding
69	10	do.	37,146	—	6	31	6	do.
70*	10	do.	37,466	—	5	30	7	do.
81	10	do.	25,958	—	4	14	18	do.
82	10	do.	29,336	—	3	13	19	do.
83	10	do.	27,056	—	2	12	20	do.
84	10	Two-course**	36,039	—	3	11	21	do.
85	12	do.	39,041	5	4	10	22	do.
86	12	One-course	38,116	4	5	9	23	do.
87	12	Two-course	38,062	3	6	8	24	do.
88	12	do.	44,782	2	7	7	25	do.
89	12	do.	44,783	1	8	6	26	Membrane curing
90	12	do.	44,189	1	9	5	27	Ponding
91	12	do.	38,094	1	10	6	28	Membrane curing

*A small portion of this mile was asphalted.

**Except for experimental lengths.

APPENDIX 2

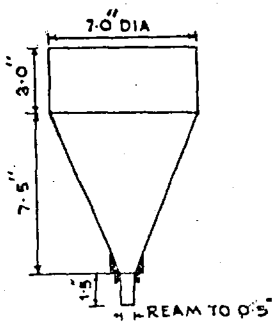
Particulars of Experimental Concrete Lengths laid on the Agra-Bombay Road

Experiment No.	Date of Laying	Particulars	Length in ft.	Centre thickness in in.	Cost per 100 sq. ft.	Remarks of inspection on 23-5-46	Remarks of inspection on 4-4-52
					Rs As.		
1	29- 9-45	BONDED CONCRETE 3 in. unreinforced concrete keyed to existing water-bound macadam base	264	3	42 12	In sound structural condition. Surface very good. Transverse crack in 6th slab $6\frac{1}{2}$-ft from dummy joint.	Condition same as previously reported.
2	1-10-45	BONDED CONCRETE Same as 1 but thickness 4 in.	264	4	48 8	In sound structural condition. Surface very good, no sign of any crack.	Condition same as previously reported. A small patch in 7th slab covered with asphalt.
3/A	23-10-45	CEMENT MACADAM (SANDWICH) WITH PRE-MIXED WEARING SURFACE (a) Base course: $3\frac{1}{2}$ thick sandstone macadam with $1:2\frac{1}{2}$ mortar sandwich, rolled. (b) Wearing coat: $1\frac{1}{2}$ in. thick premixed concrete $1:2:3\frac{1}{2}$ with trap aggregate of $\frac{3}{4}$ in. max. size	264	5	—	Dense, well-knit surface noticeable. Corner load break at beginning of experimental length; adjoining road is tarred. Edge patch in 6th slab shows the place from which a test piece was removed.	Condition same as previously reported.
3/B	27-10-45	CEMENT MACADAM (SANDWICH) WITH PRE-MIXED WEARING SURFACE Same as 3/A but mortar proportion for base course 1 : 3	264	5	—	Dense, well-knit surface noticeable. No sign of any crack.	Condition same as previously reported.
4/A	13-11-45	CEMENT MACADAM (GROUTED) WITH PREMIXED WEARING SURFACE (a) Base course: $3\frac{1}{2}$ in. thick sandstone macadam grouted with $1:2\frac{1}{2}$ grout, not rolled. (b) Wearing coat $1\frac{1}{2}$ in. thick premixed concrete $1:2:3\frac{1}{2}$ with trap aggregate of $\frac{3}{4}$ in. max. size.	132	5	43 15	 Dense, well-knit surface noticeable. Cracks as shown in sketch in 4th slab, marking end of day's work.	Condition same as previously reported. Cracks sealed with bitumen.

APPENDIX 3

Fluidity of Grout and its Relations to the size of aggregate

The flow cone is a round funnel of dimensions shown in the Fig., and has a volume of 221 cu. in. The amount of water necessary is determined by trial batches using the selected sand in the specified proportion with cement. Sand may be used just as delivered at the mixer, if a check on the actual water cement ratio is not desired. More useful results are obtained, however, if the sand is tested in a saturated surface-dry condition. To obtain this condition, the sand is soaked in water for at least an hour and then spread out in the sun in a thin layer and turned frequently until the sheen of moisture disappears from the surface of the particles.



For a given mix, about 5 lb of cement and the proportionate (weight) of sand are measured out for the trial batch. To this is added a quantity of water less than will actually be used in making the grout (say 6 gallons of water per cwt of cement) and the flow time taken.

The method of making the fluidity test is as follows : the flow cone is rigidly supported with the top level. The bottom of the ½ in. tube is closed. The sample of the test grout is brought in a pan or pail and is vigorously stirred as it is continuously poured into the flow cone. The grout is allowed to settle for 5 seconds from the time the test cone has been filled to the top. At the end of the settling period, the ½ in. tube is opened and the time in seconds for the test cone to empty is measured by a stop watch.

Observations also show that objectionable bleeding of free water occurs just before segregation causes stoppage for decreased flow. The point at which this objectionable bleeding occurs in a grout made with any given sand and mix definitely limits the fluidity which can be safely attained with that sand and mix.

Experience has indicated the fluidity necessary to insure satisfactory flow of grout through coarse aggregates of different sizes and this may be taken as follows :

Size of aggregate, (in.)	Fluidity (sec.)
$2\frac{1}{2} - 3\frac{1}{2}$	23 – 25
$2 - 3$	23 – 25
$1\frac{1}{2} - 2\frac{1}{2}$	21 – 23
$1 - 2$	20 – 22
$\frac{3}{4} - 1\frac{1}{2}$	19 – 21

The size and grading of sand with which the required rates of flow can be safely reached with the given mix can be selected.

When a coarse sand grout be used with smaller size of aggregate and if the water content were kept low so as to prevent segregation, the grout would not be sufficiently fluid to penetrate through this aggregate, although it might be quite satisfactory for larger aggregate. If the water content were increased, the coarse sand particles would separate and be deposited in the spaces in the topmost layers of the stone, and only the most liquid portion of the grout could filter through. This would result in a honeycombed pavement.

Coarse sand grouts are, therefore, suitable for use with large aggregates, and fine sand grouts must be used with small aggregates. It is also obvious that leaner mixes with coarse sands are more difficult to handle than richer mixes.

Enough water is added to make up $6\frac{1}{2}$ gallons per cwt and the flow time again determined. This procedure is repeated, adding $\frac{1}{2}$ gallon per cwt increments of water to the batch until the grout begins to show segregation, or until the flow time becomes constant. The flow curves are drawn plotted, see Plates I and II. The quantity of water needed per cwt of cement for the required flow-time can be read from the curves. If surface-dry sand was used in the test batch, the amount of water taken from the curve must be corrected for water contained in the sand on the job.

The amount of moisture in the sand varies from hour to hour; it is therefore important to check the fluidity of the grout at frequent intervals.

Plate I shows curves of flow time for three usual sand gradings—coarse, medium, and fine for a grout proportion of 1 : 2 by weight. Plate II shows curves for the medium sand but with grout proportions of 1 : 2, 1 : $2\frac{1}{2}$, and 1 : $2\frac{3}{4}$ by weight. These

curves show how the mix, size and grading of sand, and amount of mixing water affect the fluidity. It can be seen that within certain limits the grout becomes more fluid (measured by decrea-ng flow time) as the amount of mixing water increases. Also, or the same fluidity, more mixing water per cwt of cement is equired for the fine than for the coarse sand grouts. Further at there is a point where additional mixing of water increases uidity but little and still more water causes a stoppage or ecreased rate of flow with the coarse sand grouts. This is ue to segregation resulting from the inability of thinner pastes o hold the larger sand particles in suspension. A similar effect s not evident for the fine sand grout.

APPENDIX

Suggested Specification for a Cement Grouted Pavement

(The following brief specification is presented here in the hope that it might be of assistance to engineers who are planning a cement grouted road project. It is expected that such changes will be made in the specification as will make it apply to the particular job for which it is written.)

Description

(1) The pavement shall be constructed to a centre thickness of...inches by the cement grouting process on the prepared subgrade to the correct lines and levels.

(2) Coarse aggregate shall be spread between the side forms, care being taken to ensure uniform grading and cleanliness. The grouting operation shall then be performed. The pavement shall be finished with one inch thick premixed surface course applied to the grouted slab.

Materials

(3) **Coarse Aggregate**. Coarse aggregate shall consist of broken stone having clear, hard, tough, dense, durable, uncoated particles. Coarse aggregate shall be uniform in quality and free from flat, thin or elongated pieces, dust, clay, loam, organic or other soluble or deleterious substances, and shall be well graded within the following limits:

Passing	$2\frac{1}{2}$	in.	sieve	100	per cent
,,	2	,,	,,	95-100	,, ,,
,,	1	,,	,,	0- 15	,, ,,

(4) **Sand.** Sand shall consist of hard, tough, durable, uncoated grains, uniform in quality, and shall contain no dust, lumps, soft or flaky particles, shale, salt or alkali, and not more than a total of four per cent by dry weight of clay, loam, silt or mica, and shall be so free from organic matter that when tested in accordance with the standard method of test, the solution shall not be darker than the standard colour solution. The grading limits for the sand shall be as follows:

Passing	No.	7	sieve	95–100 per cent
,,	,,	14	,,	60– 80 ,, ,,
,,	,,	52	,,	10– 30 ,, ,,
,,	,,	100	,,	0– 5 ,, ,,

(5) **Portland Cement.** Indian Portland cement conforming with I.S. 269 : 1952 shall be used. Bags shall not be opened until the cement is to be emptied into the mixer.

(6) **Water.** The water used for the work shall be clean and free from oil, acid, alkali, and organic or other deleterious substances.

Construction

(7) **Side Forms.** Side forms consisting of planks, one and a half inch thick and having a depth equal to the finished thickness of the slab shall be placed carefully to line and level on each side of the pavement to form a boxing, in which to spread the coarse aggregate. The forms shall be supported firmly by stout stakes and shall remain in place for at least 24 hours after pouring of the grout.

(8) **Subgrade.** The subgrade shall be constructed to the grades shown on the plans. The surface of the subgrade shall be uniform, well compacted, and free from loose material. Any soft, spongy or yielding places in the subgrade shall be corrected before the coarse aggregate is spread thereon. The subgrade shall be well drained at all times.

(9) **Spreading Coarse Aggregate.** Coarse aggregate shall be spread between the side forms and levelled by means of a template to within one half inch of the finished surface of the slab. If for any reason the coarse aggregate has become dusty it shall be watered.

(10) **Joints.** Transverse joints of the dummy groove type shall be made......feet apart and three quarter inch thick expansion joints at......feet apart.

(11) In the event of stoppage due to abreak down of the machine, a separator must be placed at a sufficient distance ahead to ensure that no grout has flowed beyond it and grouting of the pavement upto such emergency separator shall then be immediately completed by hand mixing the grout, using fifty per cent additional cement in an ordinary mixer. Every care shall be taken, however, to avoid such stoppages.

(12) Care must be exercised at all times when operations are resumed that no damage is done to completed work, and that all surplus grout is removed before it has commenced to set.

(13) **Grouting.** After completion of the spreading and levelling of the coarse aggregate, it shall be grouted in the following proportions:

Portland cement	1 part
Sand	$2\frac{1}{2}$ parts

Mixing shall be done in a tilting drum concrete mixer. Sand shall be gauged accurately by measurement at the site or at any other approved point. For convenience both for cartage and handling at the mixing machine sand may, after gauging, be bagged in cubic feet quantities in approved bags. If sand is placed in bulk along the road-side, measuring boxes with handles holding the correct quantities for a bag batch shall be used and the sand tipped direct from them into the loading hopper.

(14) Grouting shall be started always from the highest points and then towards the lowest point alternately, and no grouting shall be done more than 8 feet ahead of any ungrouted portions.

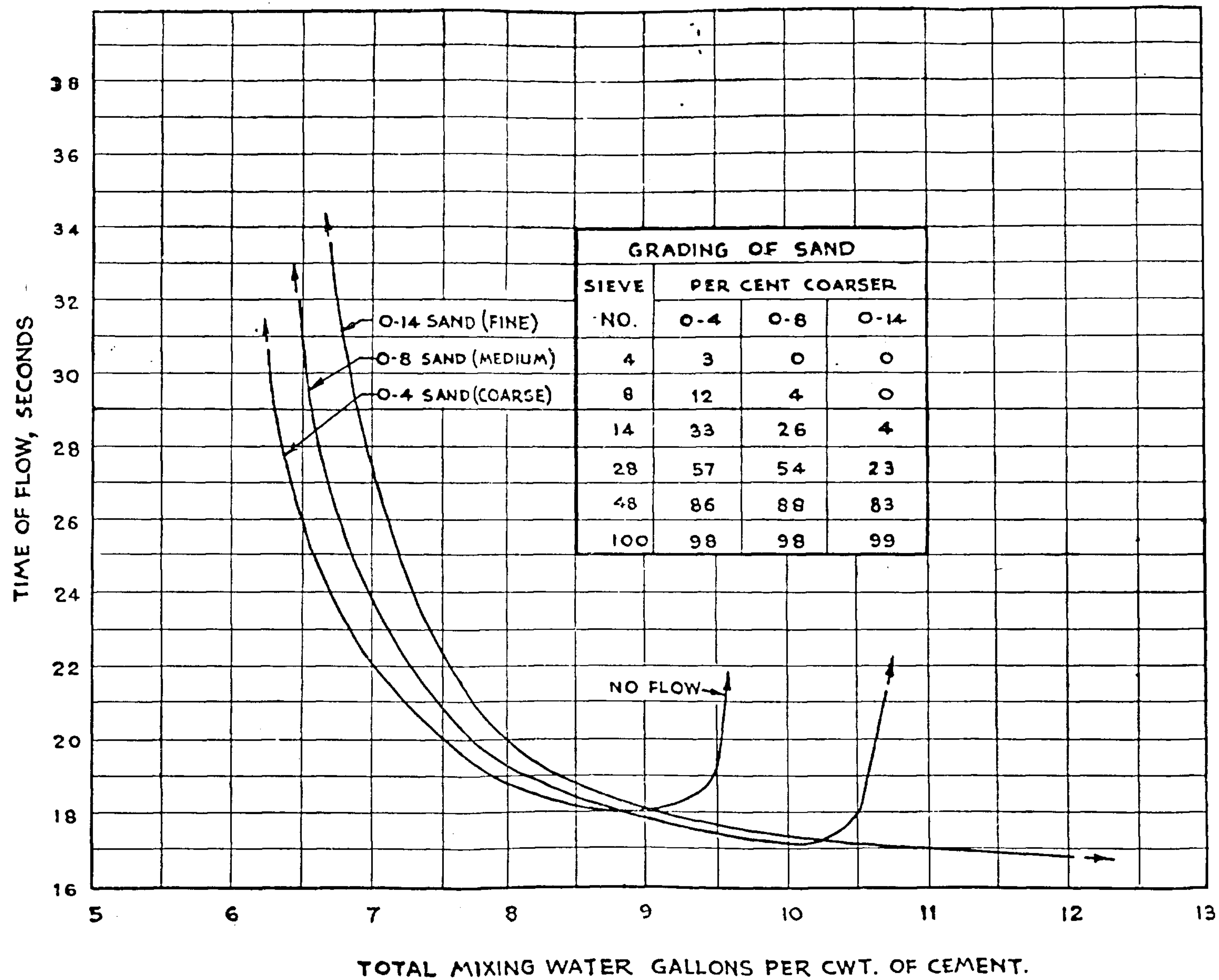
(15) The grout shall be poured until all voids and interstices in the coarse aggregate are filled and until there is a slight excess of grout on the surface.

(16) **Premixed Surface Course.** Immediately after the grouting, a concrete mix of 1 part cement, 2 parts well graded sand and $3\frac{1}{2}$ parts of broken stone graded from 3/16 in. to $\frac{3}{4}$ in. using minimum amount of water shall be placed and tamped. The finishing shall be by a belt to the required degree of smoothness. Care shall be taken to see that the water-cement ratio shall be as low as possible. Care shall also be taken to see that a perfect bond is secured between the grouted base and the premixed top.

(17) **Curing and Protection.** As soon as the surface is finished it shall be shaded from the heat and direct rays of the sun by means of gabled screens.

(18) As soon as the concrete is set sufficiently hard to prevent any marking, the surface shall be spread with double hessian cloth and sprinkled with water to keep it continuously wet until the next day when the surface shall be covered with wet earth or water ponded upon it for at least 10 days. After a further ten days all curing material shall be removed, the surface of the concrete thoroughly cleaned off and joints filled. The pavement shall not be opened to traffic until 30 days from laying the concrete.

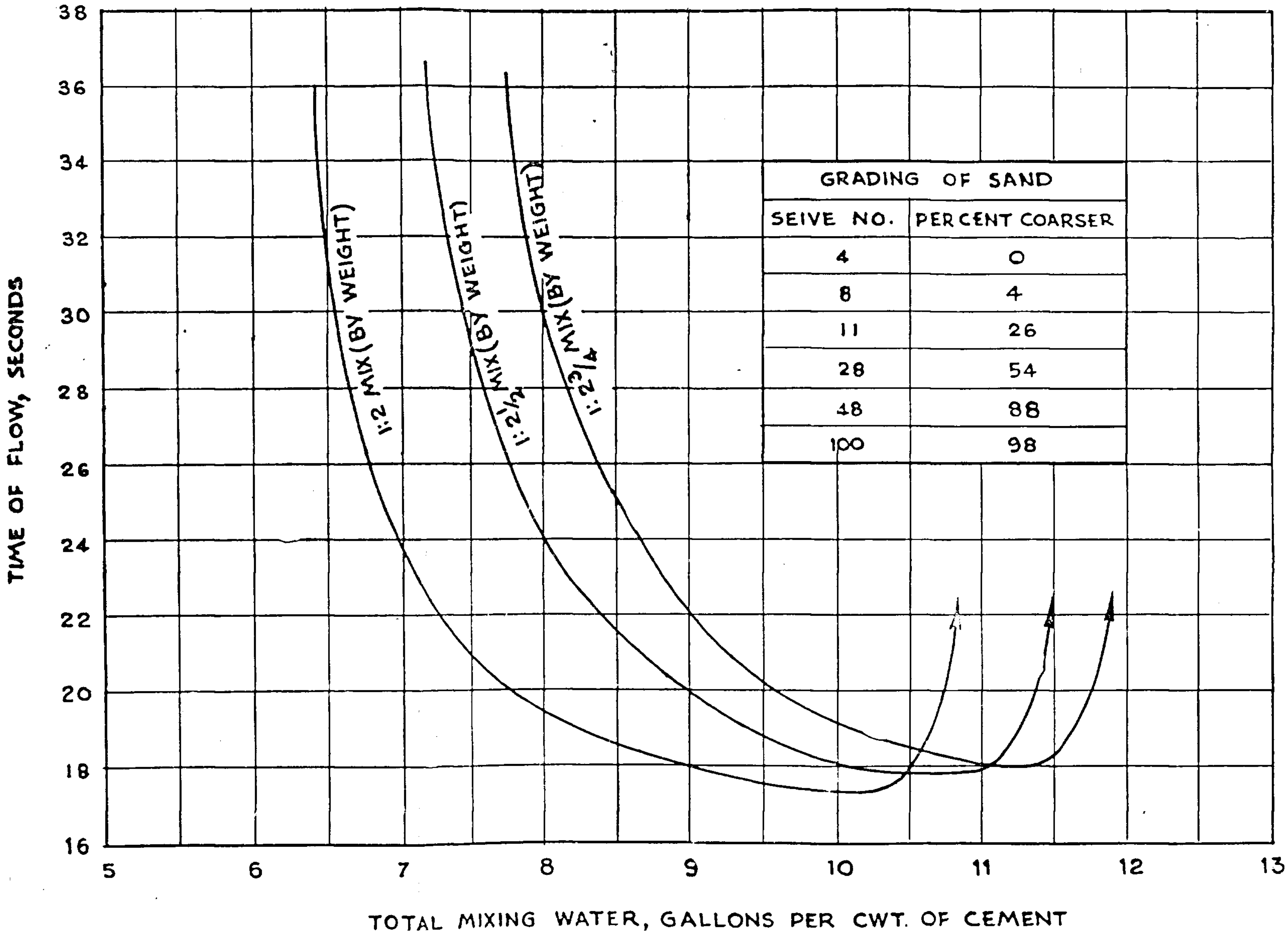
PLATE I



SIEVE NO.	PER CENT COARSER 0-4	0-8	0-14
4	3	0	0
8	12	4	0
14	33	26	4
28	57	54	23
48	86	88	83
100	98	98	99

CURVES OF FLOW TIME FOR COARSE, MEDIUM AND FINE SAND FOR A GROUT OF 1:2 BY WEIGHT

PLATE II



CURVES OF FLOW TIME FOR MEDIUM SAND FOR VARIOUS PROPORTIONS OF GROUT

Paper No. 176

RIVERS AND FERRIES OF NORTH BENGAL

By

R. P. CHANDA, B.E.

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SYNOPSIS

The Paper describes the characteristics of Rivers in North Bengal. An account of the changes in the course of the Torsa near Cooch-Behar and the Teesta near Mandalghat is given. Some of the important ferries across the Kalyani, Gadadhar and the Sil-Torsa are described through the various stages of their development. The difficulties of operating ferry services and a few accidents are narrated.

1. INTRODUCTION

1.1. North Bengal comprises of the three districts of Darjeeling, Jalpaiguri and Cooch-Behar. Road and rail routes to Assam from the rest of India pass through this tract.

1.2. Darjeeling district is covered by lower ranges of the Himalayas, except for a small area around Siliguri known as the 'Terai'. Jalpaiguri district west of the Teesta is plain but the part of the district east of the Teesta and adjacent to Bhutan Hills known as 'Duars', is foothill country. Cooch-Behar district is plain, flat and alluvial. Thus, in North Bengal the landscape is rarely monotonous and wide variations exist in the terrain. The area is mountainous in the north, has extensive reserve forests and rolling tea garden lands in the middle, whilst in the south there exist vast expanses of jute, tobacco or rice fields, interspersed with slightly higher grazing lands. The drainage basins are unstable owing to their liability to scour in the upland reaches and to silting up in the middle and the plain regions.

1.3. The three rivers, Teesta, Jaldhaka and Torsa, with their numerous tributaries dominate and ultimately join the Brahmaputra. The Mahananda on the western border and the Kaljani, Gadadhar and Raidak on the east drain the Siliguri and Cooch-Behar areas respectively into the Ganga and the Brahmaputra.

1.4. The Teesta has its source near the Zemu glacier near Kanchanjunga in the inner Himalayas, and flows through the hills with a gorge section. It enters the plains just above the recently constructed railway bridge on the Assam rail link and fans out in a broad and comparatively shallow stream forming the wildest and the most devastating "river of sighs" in North Bengal. Jalpaiguri on its right or west bank is the headquarters of the district, most of which lies on the east side of the river. There are two road bridges in the hilly section but none in the plains. There is a tendency for the river to gradually raise its bed in the plains. This is due to the deposit of silt brought down annually from the hills. Within its defined banks the flow is usually divided into several shifting channels.

1.5. The Jaldhaka (known as the Mansai or the Dharla in the lower reaches) is the main stream of the central basin. It often breaches its flood banks and causes extensive havoc on the Assam Access Road (National Highway No. 31). The main river and all its affluents except the Diana, have been bridged.

1.6. The Torsa has its source in Tibet and drains the Chumbi valley besides extensive areas of foothill country and the plains. Due to the instability of its flow channels and its tendency to break out and join adjacent channels in the valley, there have been so many changes in its course during the last 15 years that the pre-war maps no longer depict the present course. It is only for short lengths that its several courses join together to form a single channel.

1.7. The basins of these three rivers often merge into one another during high floods when overflows pass from one to the other across the low blurred ridges. Hence, fixing of bridge ventways and design of protective works for these rivers or their tributaries is difficult.

1.8. Rainfall in the area is heavy, exceeding 140 inches in a year and heavy downpours are of more frequent occurrence than drizzles or light showers. After every spell of torrential rains in the hills and in the upper reaches of the river, a flash flood rushes down the valley. The flood season starts from June and continues till the middle of October.

2. CHARACTERISTICS OF THE RIVERS IN THE FOOTHILL AND PLAIN REGIONS

2.1. The regimes of these rivers are extremely unstable and their behaviour is unpredictable. Roughly, they can be divided into the following three types :

(a) More or less perennial rivers in the plains flowing in deep or shallow channels over wide or narrow beds and between banks more or less defined. These show considerable variations in the depth of water from season to season and from day to day in the flood season.

(b) Broad, shallow rivers in the foothill regions with their beds not much below the level of the adjacent ground. These carry very little flow at normal times but get swollen during floods for a few hours or for a few days. The flood waters are heavily silt laden and roll down lot of shingle and boulders from the hills. For a greater portion of the length, the courses are not well defined and are meandering in the extreme.

(c) In between the above two types, there are numerous streams which are fordable in dry season but not in flood season.

2.2. The flow of these rivers changes from slow and sluggish to a swift current, within a short interval of time. River beds have slopes of about 50 ft per mile in the upper region and 2 to 3 ft per mile in the lower region.

2.3. Deep and stable river sections are rare. Even in the upper regions where the beds are stable, bed scoursoccur during floods but they get filled up insubsiding floods. In a broad river, a deep section may exist but it will be narrow and shifting. Ferry operation is possible during rainy season in the plains.

During the flash flood season, however, no ferry service is either possible or safe.

2.4. Water sheds for the different streams and rivers can hardly be distinguished. In the mid-zone, the rivers have the characteristics of the upper and the lower reaches mixed up in a way so as to make navigation a troublesome and risky venture. In a flood, sudden rise in a river may be as high as 6 to 8 ft. Maximum rise above the dry weather flow might be 15 to 18 ft in the lower region and much more in the higher region. In falling floods, silt and debris deposits will be heavy and shoals will be formed and "*chars*" or high beds will be exposed. In the dry season, there will be long and dry sandy beds with narrow and sluggish streams.

2.5. Even in high floods, a river, may have wide shallow strips flanking the deep water channel and the flow may be divided into 2 or more parallel streams within defined banks. Pure clay soil is a rarity in beds or in banks. Fine sand or silty soil commonly found has little cohesion and is easily erodible.

2.6. The flood in the rivers may be retarded and headed up if the main outfall (the Brahmaputra) is also running high at the time, and then there will be broad sheets of water flowing sluggishly through the valley and submerging the fields. In the foothill regions, temporary obstructions in the course of a rapidly flowing stream, such as some very large boulders or trees partly bedded, or apart of a forest may cause a serious deflection in the course and divert a part or the whole of flow into a different stream. One frequently comes across such "dead courses" in this part of the country.

3. EXAMPLES OF CHANGES IN RIVER COURSES

3.1. Some recent changes in the regime of important rivers illustrating the peculiarities discussed are described below.

3.2. The Torsa near Cooch-Behar

In the lower reach of the Torsa above Cooch-Behar town, after different tributaries have joined the main river, there had occurred a cut-off in the neck of a 2-mile long loop in September 1951. At the base, the flow was deflected by nearly 90 degrees. Since then, during high floods the overflows moved over low land deepening and widening abandoned river beds and depressions in the way. The overflows then entered into a minor river in the vicinity and rejoined the main course down below. In July 1953, the river forced its way through this minor stream which was only 250 feet wide. As a result, the diverted course got widened to over 700 feet in a few days. The new diversion got about 50 per cent shorter in length than the abandoned course and the current is very swift even in low waters.

3.3. The Teesta near Mandalghat

Near Mandalghat, much flow from the Teesta was getting deflected, during the last 10 to 12 years, into an old and almost dead branch, the Buri-Teesta, in gradually increasing proportions. The channel of the Buri-Teesta was lately getting enlarged. The 120 ft span railway bridge across the river acting as a bottle-neck, impeded free flow and prevented the main flow from getting diverted. But the flow had broadened out into a 600 ft wide channel below this bridge and the banks were overtopped during floods. The flow dammed up by the railway bank for several miles had caused considerable damage to the countryside and also to road banks and bridges. By restricting the flow successfully, railway bridge had caused, by the end of 1952, the silting up of the deep channel of the Buri-Teesta between its offtake and the railway bridge.

3.4. Where a river follows a fairly defined course and swings within limits, training or guiding may be possible at the particular section. But, the conditions in this area are such that it requires protective works from the point, where a river comes out from the hills. Hence the selection of a safe and proper site for a bridge or a ferry across a river is very difficult owing to the meanders, loops, etc. which occur throughout the course. Obviously, bridge construction is unduly expensive on account of heavy protective works necessary and ferry service is costly, uncertain and risky.

4. GROWTH OF TRAFFIC AND DEVELOPMENT OF FERRIES

4.1. It may appear strange that river ferries should be an important means of communication in this area which is drained by many rapidly rising and fast flowing rivers. Besides major rivers, many important tributaries have enough flow even in the dry season to render ferry services indispensable. Before the World War I, vehicular traffic in this area was little and was mainly confined to the feeder roads from tea gardens, forest depots, and market centres to the railheads. Railways were the only means of long distance transport.

4.2. After World War I, however, road construction across thick jungles and crossing many rivers and streams, was taken up on a fairly large scale in the *Duars* and *Terai* areas. In fact, this was the first developed highway network in undivided Bengal to serve the growing needs of tea gardens, and areas cultivating jute, tobacco and timber.

4.3. The Jaldhaka river was bridged in 1929 and the Teesta, near Sevoke, in 1941. But a through all-weather road was not there. In Cooch-Behar, which was a seperate State, there was hardly any motorable road.

4.4. Bullock carts and light motor vehicles ferried across rivers were the chief means of transportation. Only the few river crossings, referred to later, were used by heavy motor vehicles. Since the rivers were all generally too shallow for the greater part of the year and admitted only of small and light boats drawing small depth of water single ferry boats requiring deeper draughts, as found on the South Bengal rivers, could not get across any of these rivers. Hence a lightly built double boat craft or '*mar*' was adopted. A raft of strong bamboos in one or two layers was built over two boats, with a gap between, for stability. Usually one or two layers of split bamboo mattings, known as '*hadla*' or '*chegar*' were next laid over a layer or two of full bamboos to form a platform. Though such a '*mar*' could carry up to six loaded bullock carts at a time, the platform could not support the concentrated wheel loads of a truck having the same gross weight. Where heavy vehicles were ferried, strong wooden platforms were provided.

Improvements to such crude ferry services were taken up during the latter part of World War II when a trunk route to Assam was being established across North Bengal, utilizing some of the existing roads en route. The improvements consisted of use of bigger boats, wooden platforms instead of the bamboo ones, provision of drop gates for landing etc.

4.5. After the war, there appeared on this direct trunk route more and more of light and heavy motor vehicles engaged in long distance passenger and goods traffic, due to the rail routes being too circuitous and slow. The importance of this route increased after the partition when the three North-South rail routes were cut off, and the Assam Access Road became the sole line of communication between Assam and the rest of India. Until

the Assam rail link was opened to traffic in 1950, ferry services on this road had special importance.

4.6. Even now, in spite of bridges across some of the rivers and the rail link, ferry traffic is considerable. In fact, ferries will always be an important feature in this area as construction of bridges at all the crossings will be costly. Besides, crippling damage to drainage structures at several points is almost an annual event during the flood season, and when roads are cut off, ferries on diversions have to be resorted to, Photo 1.

5. GENERAL PROBLEMS OF FERRY SERVICES

5.1. Since the rivers in the area are subject to the vagaries and peculiarities referred to above, regular and efficient ferry services cannot be assured whatever might be the arrangements. However, the several problems have been tackled in the following manner.

5.2. Dry Season Working

Shallow Depths and Shoal Formations. Whether the waterway is narrow or wide and whether it has a deep channel or not, always will be found inconveniently shallow waters for some portion of the width. So, boats have often to take circuitous routes and loads have to be restricted. In the smaller rivers, it may be possible to improve the channel somewhat by narrowing it by means of earth bunds.

Temporary Bridge. A dry-weather bridge is generally constructed if the channel is not too wide by hand-planting sal '*ballahs*' in the bed and often by using such '*ballahs*' as beams to support deck planks. The trestles supporting the beams consist of 3 or 4 posts and are generally spaced about 10 ft apart. The decking may consist of sal '*ballahs*' nailed to beams or of full bamboos and '*chegar*' matting for use of lighter vehicles only. In due time, before the onset of the monsoon, these are removed and used again in the next season. Such constructions cost about Rs 30 to 40 per foot.

Fording of Streams. The sandy bed of small fordable streams may be stabilized by dumping stone shingles or rubbish.

Approaches to Ghats on Sandy Beds. Where the bed is completely dry and sand is deep, the track is usually stabilized by spreading grass fibres of sufficient thickness. Bullockcarts are kept out of such tracks, as cart wheels cut up the grass surfacing. 'Kashia' grass which is available in the upper regions, is fairly tough and useful for the purpose. But it has to be carted over a long distance for use in the lower reaches. Where no improvement is practicable, only vehicles with 4-wheel drives could use the bed. On river beds covered by large boulders, loose or bedded, the riding surface is provided by spreading and dressing natural gravel and sand mixes from time to time. Laying continuous hard tracks has not been tried yet, because of the high initial costs and availability of long grasses in the upper region.

Bank Slopes. In crossings used by heavy vehicles, ramps cut through banks should be stabilized with river shingles and should not be too steep. Many accidents have occurred in which the vehicles, after going down the slopes, have overshot the boat platforms and fallen overboard.

Boat Bridges. These are not common for the following reasons:

(1) non-availability of boats in sufficient numbers for the purpose,

(2) rapid current in the higher regions, and

(3) care of boats, essential repairs for them and keeping watch are expensive.

Yet there could be several such bridges in the lower reaches, in the dry season, if light and flat bottom metal pontoons are available.

Sudden Changes in water level. The rivers suddenly become too shallow for a part or full width of their waterway, at the established crossing. In such cases dry-weather arrangements have to be scrapped. Shifting the ghat and load restrictions may be necessary at this time. Such difficulties are also felt when snows in the hills begin to melt or when there is an early or unseasonal spell of rains.

5.3. Wet or Flood Season Workings

The difficulties of the ferry operation during flood season are due to:

(a) the flow being too swift;

(b) shallow waters on the approaches due to overflow;

(c) the river flowing through a number of streams within the defined banks;

(d) floating or submerged trees;

(e) sudden variations in water level; and

(f) silting up of the approaches.

Care and attention of Water-Crafts. Boats are liable to be lost due to one or the other of the following and often the boatmen have to keep night-long vigilance during difficult periods:

(1) When anchored in deep water they might be forced away during high floods.

(2) The tree to which a boat is tied up may be uprooted.

(3) Sudden collapse of the adjacent bank on the boat may sink it.

(4) The boat may be sunk in a deep scour hole formed around it and then quickly covered by fresh silt or sand deposit.

(5) If a boat is anchored on the shallow side during the peak flood, it may be left high and dry in the course of a night when the flood subsides. It is not possible to move a *mar* on the dry bed without dismantling the platform.

So care of boats during idle periods can only be entrusted to experienced boatmen.

6. TYPICAL EXAMPLES OF RIVER CROSSINGS*

6.1. **Ferries across the Kaljani, the Gadadhar and the Raidak on the newly aligned Cooch-Behar-Buxirhat (Assam Border) section.** The particulars of the boats used are given in Appendix 1. The difficulties in the operation of this ferry when it had to take heavy vehicles are given below.

When a fully loaded truck was just on the point of getting off, the loading boat dipped almost to the water level or a

* This section is published in an abridged form. The original contains elaborate description of ferries. Members interested can obtain a copy from the Secretary, Indian Roads Congress.—*Ed.*

little below under the weight of the rear wheels. As a result, splashes of water got into the boat every time a heavily loaded truck came down, and had to be baled out. If, by chance a loaded truck stopped due to engine failure with the rear wheels on the outer edge, the forward boat would even capsize.

The ghat is so built that the drop gates or ramps would always slope down to it. When the front wheels of a loaded truck just got on to the boat, the hinged ends of the gates would go down by about 6 inches. If the ramps were in level initially, its free end would then rise higher and clear the ground. In that case, the rear wheels of the truck got blocked by the ramp itself. Then the ghat level had to be adjusted before the truck could get on to the boat.

Sometimes failure of O. B. engines occurred when boats would drift down almost uncontrolled. Soon, further improvements to ferry services were felt necessary in the interests of long and regular convoys moving to and from Assam. In July 1949, six aluminium bridging pontoons, with usual components, details in Appendix 2, were procured. Two such pontoons could be fitted up as a raft, in a day, and could be dismantled quickly. But these pontoons had square noses and, therefore, could hardly move against even a moderate current. A 22 H. P. engine was inadequate to drive these pontoons against a fairly swift current.

In July 1949, 30 ft high towers, for running 'flying ferries', one on each bank of all the three rivers were erected. The towers were built with Bailey Bridge components and a 1-in. diameter wire rope was stretched across the river, with its ends passing over the towers and anchored to the ground securely. The greatest distance between two towers was about 600 feet. ½-in. diameter wire rope, with an end to be hitched to a pontoon to serve as towing cable, was attached to the suspended cable through a snatch block, which could move freely on the line, Photos 2 and 3.

In crossing a river, the towing cable was attached to the bow of the rear pontoon and the raft was made to turn at an angle to the current so that a component of the current would push the *mar* across the river at a fair speed. In high current, the raft would be prevented from shooting across by keeping a small angle to the current by use of the rudder. The rudder would also keep the *mar* steadily moving forward and prevent oscillations.

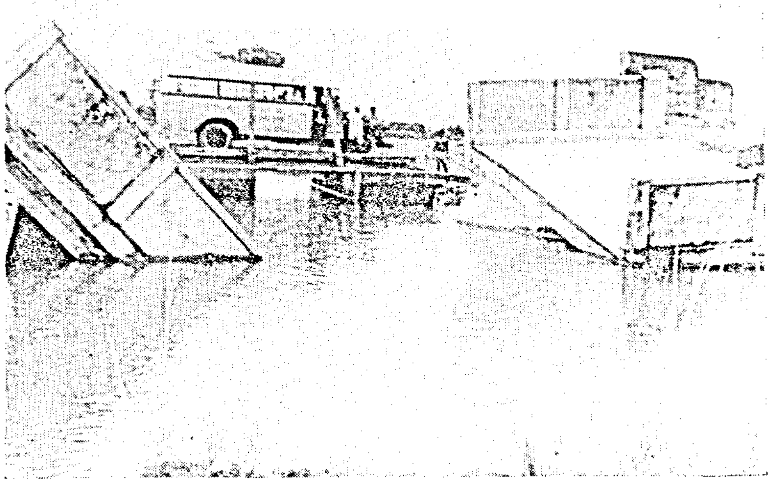
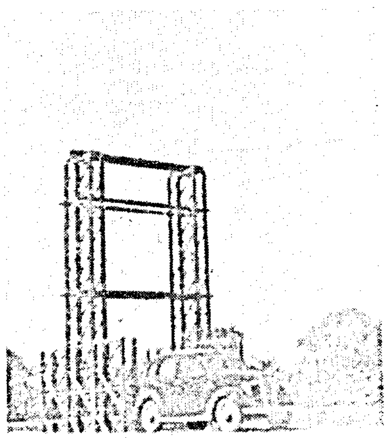


Photo 1. **Ferry operating at the site of a collapsed bridge on Cooch-Behar Baxirhat Section of N. H. 31 in the floods of 1952**

Photo 2.

A 30 ft high tower supporting the suspension cable at the Kaljani river ferry



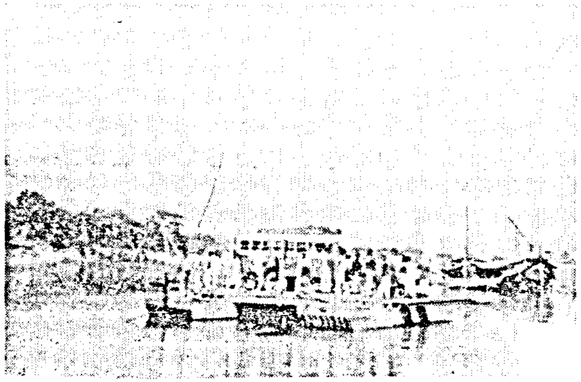


Photo 3.

Flying ferry service. A Bus is being carried over a raft of pontoons

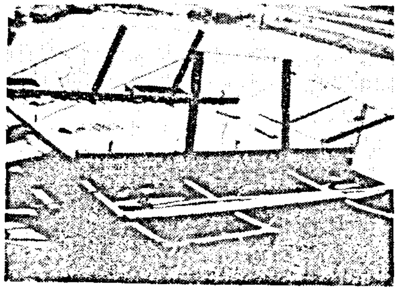


Photo 4.

Close view of aluminium pontoons, platforms and balanced drop-gates

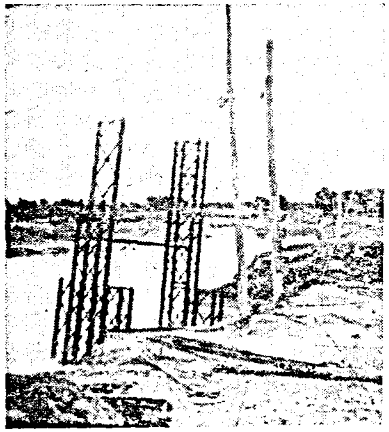


Photo 5.

The tower of the ferry across Gadhadhar being dismantled

A sketch is given in Fig. 1.

The pontoon ferry would require only 3 men to operate. During loading and off-loading of the heaviest vehicle in use, i. e., a steam road roller, pontoons were stable. As the rivers had fairly deep sections, shoals were not significant and almost straight crossings were possible. Otherwise, the towing cable would have to be adjusted so that the boat could move up or down stream.

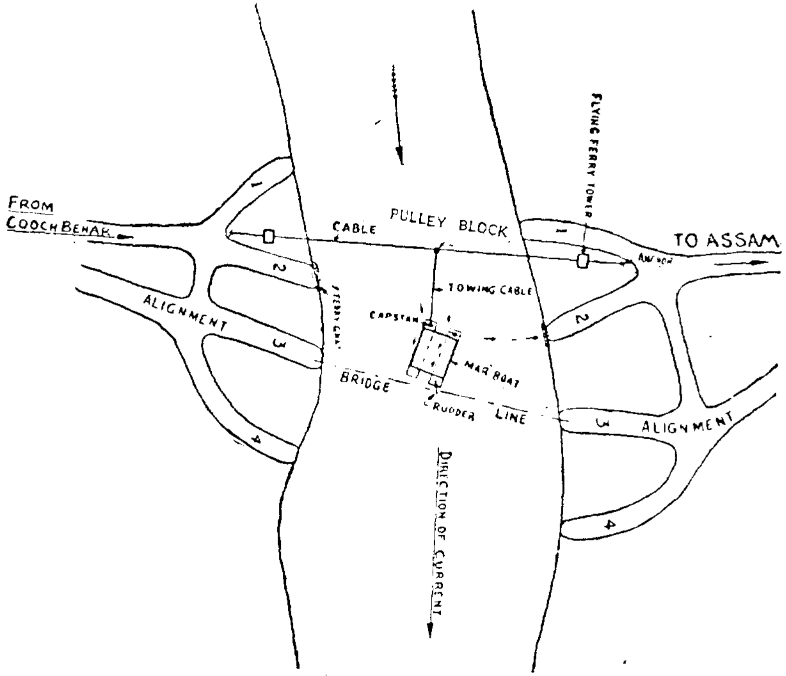


Fig. 1.

At this time, for convenience of loading and off-loading vehicles, balanced drop-gates or ramps were fitted up on either side of each raft. Details of the drop-gates are given in Appendix 2 and shown in Photo 4. The 10 ft long drop-gates were found to be particularly useful. These improved arrangements were fully brought into use from November 1949. From then onward, flying ferry arrangements were the principal means of crossing the three rivers. Until May 1952, when the new bridges across these rivers were opened to traffic, the ferries worked well and no accident occurred. In one case, a tower was badly scoured underneath and it leaned over much during a flood in

1951. This was carefully dismantled and re-erected on safer ground, Photo 5.

Occasionally, due to waiting vehicles or disabled vehicle on the approach road, there was congestion on the roadway. Due to lack of alternate approaches to ghats, traffic was held up. It was, therefore, decided that, for each crossing, the following facilities should be provided:

(a) There should be three additional approaches, starting at high level from the main road, towards separate ghats at different levels. Their surfaces were to be stabilised with river shingles. Thus a total of four ghats would be available on each bank.

(b) There should be 4 sets of *mars* of 12 ton capacity each available at each crossing.

(c) A large number of O. B. engines and navigational aids should be kept in stock.

These four-point ferry arrangements were completed later in 1950 but there was no occasion for their full use. The particulars of 12-ton double-boats, as rigged up by the riverside, are given in Appendix 1. For loading and off-loading, separate farashes or ramps, 7 ft 6 in. long, were provided. These heavy *mars* were not much used actually as each required six to seven men to handle. These also required high approaches or ghats. Each set without drop-gates had cost in 1950 Rs 5,900.

6.2. Ferry across the Sil-Torsa at Silbariaghat, in Falakata-Patlakhawa Section of N. H. 31

Failure of existing Bridge. The bridge on this branch of the Torsa was washed away early in the flood season of 1948. At that time, this channel was drawing an increasingly large proportion of the flow from the parent river. The river here has a fairly steep slope and the current is unusually swift, with a velocity as high as 8 to 9 ft per second. Just before the failure, large numbers of uprooted trees from the higher reaches came down floating and some of these, caught by the bridge piers, blocked much of the waterway. Piles in the foundations were badly scoured and some of the piers collapsed.

Because of the swift current, two 22 H. P. O. B. motors had to be provided. These were fitted on the foreside of the *mar*. In January 1949, when the river went nearly dry, a temporary bridge of about 150 ft in length was built with timber deckings supported on welded steel cribs placed on the river bed. In May

1949, in the course of a sudden flood, with the driftwood blocking the vents, much of this bridge was washed away. The 6-ton *mar* boats were then put into service again, powered by O. B. engines.

Some Accidents. In June 1949, while the *mar* carrying a truck and some passengers was going round crossing the deep channel, it was suddenly caught by unexpected under-water obstruction and immediately its bottom was ripped open. The truck and the boats with the two engines fitted to it could not be found and 3 lives were lost. The platform of the *mar* was found however, later about 20 miles down.

In July 1949, a new *mar* was launched. At the same time flying ferry arrangements were set up. Two 40 ft high towers were erected one on each bank and 400 ft apart, near the bridge site. A pair of Bailey pontoons were also fitted up to carry the raft. But the current was very strong and the boats used to strain the towing rope and the flying cable so hard that the boats pitched down dangerously by the bow. After two days' use, these arrangements had to be dismantled and refitted about 600 ft down to avoid the construction then existing near old site. Even then the current was too much and within two or three days, the first failure occurred when the towing rope gave way while the raft carrying two empty trucks had pitched so much that water had just begun to flow in. On being set free, the *mar* promptly balanced itself and an accident was averted. During a crossing, the tower on one bank was pulled down while the raft was still connected to the flying cable. After this event, flying ferry arrangements were removed altogether.

In February 1950, a temporary timber bridge over 100 ft long was provided using hand-driven sal piles as supports. During an early flood in May 1950, this bridge was swept away. A light vehicle was caught by this flood while on the bridge. It managed somehow to leave the bridge just in time. Since then no further attempt has been made to provide temporary bridging.

Meanwhile, it was decided to shift the ferry site by about 2 furlongs downstream, where the river had narrowed down between comparatively high banks. At the bridge site, the river had then spread out too wide and the bed was high and almost flat. Approach roads with shingled surface were constructed before the floods. Towards the end of the monsoon in 1950 much of the left approach road was, however, lost by deep erosions. Hence, the ferry site had to be moved up again.

In October 1950, a pair of aluminium bridging pontoons was provided, but these could not be used much except in the dry season when the flow was too shallow for country-boats. These pontoons with their square ends would wear out power units much quicker.

In July 1952, after an extraordinary high flood, the silt depositions were unusually heavy. The left approach roads and bridges provided were breached. Pending construction of a new low-level and stable approach, the ferry service had to be suspended for several days. An 80 feet long jetty was built to help the situation but it became quickly useless because of the formation of a shoal in front.

The river is wide and has a defined channel of flow during the wet season. The ferry site lies a little below a region of stone deposits. To cope with small floods and slight rises above high beds, a low and long jetty has to be built so that approaches on soggy or submerged char lands may be avoided. In 1953, a 280 feet low jetty just above O. F. L. was constructed and further extension by another 120 feet was also constructed.

Some particulars about this ferry, such as cost of operations, volume of traffic, river characteristics, etc., are given in detail in Appendix 3. Excluding the costs of watercrafts, new approach roads, and overhead charges, the average annual expenditure during the last 5 years has been of the order of Rs 95,000.

6.3. Jalpaiguri Town-Barnesghat Teesta Ferry

The bed of the Teesta at the site of the ferry is nearly 2 miles in width. It has 3 or 4 parallel channels, besides the main stream, in the dry season and has thick growth of 'Bhabni' grass on its high bed. The ferry here acts as a funnel in passing passengers to or from the district headquarters. At Barnesghat on the left bank, many bus routes from the *Duars* converge and terminate. Hence there are always local as well as long distance passengers availing of this service.

The distance between Moynaguri road junction (5 miles from Barnesghat) and Jalpaiguri town is about 8 miles by the ferry route and about 92 miles via the Teesta bridge near Sevoke and Siliguri. Again, the distance between Moynaguri road junction and Siliguri (which is the most important communication centre in North Bengal) is 64 miles via Sevoke bridge and 36 miles via the Barnes ferry. Hence special efforts are made to keep open the vehicle ferry as long as possible. In spite of substantial savings in the distance, not many vehicles are attracted to this ferry because of the delay and interruptions likely. Hardly any

through traffic to or from Siliguri and beyond, uses this shorter route. Details of traffic at the ferry are given below:

Average daily traffic

	Dry season	Flood season	Remarks
Passengers (on foot)	2,000 No.	500 No.	There may be total stoppages in the flood season.
Trucks	10 No.	Nil.	Loaded trucks constitute half.
Light vehicles	15 No.	Nil.	...
Bullock-carts	15 No.	Nil.	...
Average daily earning	Rs 275	Rs 60 to Rs 70	Luggage is charged separately.

The ferry is worked on the main channel throughout the year while other channels are bridged in the dry season. In the fair weather, slightly raised roadways are provided on dry beds and the soil is stabilised with 2 or more layers of 'Bhabni' grass each layer being 3 inches thick. Two sets of track are usually provided, one for light passenger vehicles and the other for trucks and bullock carts. Besides, a large number of taxis on either side make quick trips on the river bed between the main stream and the river banks. In the flood season, boats are placed on separate streams, but due to shallow depths these may fail to negotiate fully all the channels existing at any time. Passengers have then to wade through knee-deep waters. During high floods, the service may be suspended for 2 or 3 days at a time.

For temporary bridges sal pile trestles, 6 feet apart, support sal wood beams to which are nailed sal *ballahs* as decking. Railings are provided for safety. In 1952, the total length of bridges provided across four channels was over 400 feet, in 1953 it was about 550 ft again over four channels. Cost of bridging is about Rs 45 per foot with new materials and Rs 20 when salvaged materials are used. These bridges may be put out of commission temporarily if there be a slight rise in water level, say after a spell of untimely rains or due to melting of the snows in the mountains.

The main stream of this river is not ideal for heavy power operations. The deep channel is often blocked, by shallows or flats.

Two types of boats are in use for passengers. There is the large open type boat which can carry a maximum of 100 men and their luggage at a time. Particulars are given in Appendix 3. The other type 'Voulia' is partly covered and provides shelter for 16 passengers who pay a higher rate.

6.4. **Toll Bridges for Vehicles**

In the upper reaches of the Torsa (near Hashimara) across the Diana (near Chengmari) and the Raidak II (above Kumargram) and in the heart of the vast tea garden areas not far from the foothills, toll bridges are usually constructed in the dry season as plying of boats is risky due to the flow being either too swift or too shallow. Besides, the river beds abound with stone boulders, shingles, and buried or submerged trees.

After the floods, the tracks are cleared and improved by spreading gravel and sand mixes to proper thickness. Their maintenance is an expensive item in case of Torsa, which has to be approached over 3 to 4 miles of the beds of abandoned courses through thick growth of tall grasses and jungle.

Crib bridges suitable for loaded trucks are constructed by the lessees. Cribs are generally triangular in shape with apex pointing upstream but these may also be rectangular in section. These are built with driftwood, large quantities of which are available on the dry bed. These may need some rough dressing, cutting to lengths and making auger holes at ends. On the bank or in shallow waters these logs are so placed as to make a triangular frame with 4 to 5 feet at base and 12 feet in height. The corners are held by passing $\frac{3}{4}$-inch diameter rods through end holes. Cribs are made high enough to stand out about 4 feet above the water level. This type of bridge is constructed in rivers running up to 5 feet depth of water. To move these into the swift flowing stream and to keep them in proper position (about 15 feet centres) the help of an elephant may even be taken. Immediately after putting in position, the insides are filled in with boulders. A 10 feet road way consisting of *sal ballah* pieces fixed to 5 or 6 lines of driftwood logs placed as beams. is constructed Railings are provided and deckings may be improved by spreading gravel and sand mixes. This type of bridge costs from Rs 40 to Rs 50 per foot length.

6.5. For a temporary ferry across Gadadhar near Golaganj the '*mar*' was powered by a Ford V-8 engine removed from a truck and fixed to a platform built in the space between boats near the stern. To the driving shaft of the engine, kept above the high water line, was connected special transmission arrangements for driving the propeller screw in water. As the engine block weighed about one ton, it could not be fitted up on the fore-part without causing an undesirable slant. When it was placed near the stern, boats were kept on an almost even keel. For this installation, a commodious shelter was provided just clear of the rudder. To get a clear vision, the rudder man had to take his position on the roof of the shelter.

The engine had to run on low gears most of the time, as in top gear, with a low cruising speed, the battery would be getting discharged. To eliminate its defective radiator, the cooling water was made to pass through an oil barrel cut open and duly connected to the engine. When the *mar* was empty, the highest point of the propeller blade would be about 8 inches below the water level. The single piece screw blade used was 15 inches tip to tip. Hence the lowest point of the blade would be nearly in level with the keel of the boat. The propeller would go down as much as the boats on being loaded. The screw blade can be lifted up by about a foot, by means of a lifting screw fitted to the base, for inspection purposes. The engine base had been so constructed that, under full load, it just touched water level and there might be slight flow over the platform. The exhaust pipe was kept a little above the high water line.

7. POSSIBLE IMPROVEMENTS

7.1. Ferries in this area will continue to play for long its present role. Hence, improvements by way of better watercrafts, approach roads, ghats, loading arrangements, and navigational facilities with power drive are necessary.

Of all the improvements now under trial, the most popular is the power unit used by the departmental ferries run on the National Highway. Several ferry farmers who are impressed about its utility have gone in for them. The prospect of cutting down the time of crossing appreciably and the increased safety are the attractions. As an example of saving in time, it may be cited that the Mansai near Gossanimari could be crossed on a '*mar*' driven by a 16-H. P. motor in 10 minutes as compared to one hour required to take a circuitous route, involving the crossing of two separate channels and wading through knee-deep waters between the two ferries. Unfortunately, high speed O. B. engines wear out quickly and timely repairs become almost impossible for want of spare parts.

7.2. In-board engines are undoubtedly more reliable and durable and it seems a good idea to utilise an over-hauled automobile engine for the purpose. But boats have to be little heavier and a minimum depth of over 3 feet has to be maintained for the whole width. The only site in North Bengal where such an arrangement can be used and that too with occasional interruptions, seems to be the Jalpaiguri-Barnes Teesta crossing.

7.3. **Temporary Bridges.** Many rivers and streams are fordable in the dry season and some in all weather, but their beds may not be always capable of sustaining heavily loaded trucks. So more temporary bridges are a necessity. On stony river beds, pile driving may be difficult and slow. Where the arrival of a flood can be anticipated well in advance and salvaging could be done in time, there is scope for use of light steel cribs to support bridge deckings.

7.4. **Boat Bridges.** Pontoon or boat bridges can be put up at suitable sites even though rarely done at present by narrowing the waterway sufficiently to get a proper depth of water. But these require aluminium or steel pontoons.

7.5. Use of light steel or aluminium boats will be an improvement on country made boats, because for the same capacity, these will draw much less depth of water.

7.6. Use of sal planks made out of old sleepers for trackways over the sandy approaches to ferries will facilitate easy travel. These may be tried where suitable grass are not available near about. But these have to be constantly attended to and arrangements have also to be made for their prompt removal before an expected flood.

Kaljani Ferry on Cooch-Behar Assam Border Section of National Highway 31

1. **List of Components and Accessories for a Double Aluminium Pontoon Raft, 27 ft long and 15 ft wide**

(a)	25-ton bridge pontoon, Model 1940, each weighing about 1 ton ...	2 No.
(b)	Balk for decking— 8 in. × 6 in. × 22 ft (space between pontoons is 15 ft and balks are staggered to get the length of 27 feet) In 8 lines ...	12 No.
(c)	Wheel guards or Ribands ... (Pontoon balks were used for the purpose)	2 No.
(d)	Chesses, full size— 12 in. × 3 in. × 15 ft for decking ...	27 No.
(e)	Buttons or chips, between chesses ...	52 No.
(f)	Clamp assembly side rails, to fix ribands to deck balks ...	10 No.

2. **Details of Balanced Drop-gates, each 10 ft × 2 ft 6 in.** (Photo 4)

A drop-gate consists of 4 No. of 5 in. × 3 in. × 10 ft 6 in. scantlings on which 1½ in. thick planks are fixed with nails and bolts and nuts. Hinges consist of 1½ in. M. S. rods, 3 ft long each, with slots for ½ in. diameter safety pins.

Gates over two sides of a raft are connected by 5/8 in. wire ropes passing over 6 in. × 6 in. × 8 ft sal wood uprights and fixed with bulldog clips. Each upright is fixed to a riband and also to an outer beam under decks, with two bolts and nuts. Near the hinge, a wedge shaped piece of wood or ramp is fixed with nails on each gate, so as to allow smooth riding to the pontoon deck. Two gates on either side are lashed to each other to maintain the gap and to act together. To keep the wire ropes in position and to prevent damages, U-shaped clamps were fixed on top of the uprights.

3. **Details of a Flying Ferry** (Photo 2)

Erection of a 30-ft tower. On a level ground, 2½ in. chesses of Bailey bridges are first laid on which 5 or 6 plain stringers are placed to come under each tower leg. On this base, 3 pairs of Bailey panels 10 ft high are made to stand on each side. On top two pairs, one above the other, are fitted up to make a total height of 30 feet. The two legs are braced together with transoms and sway bracings. Each pair of panels was braced by means of bracing frames.

Towing Arrangements. A 1-in. diameter wire rope was used as the suspension cable across the river. After passing over the towers, ends were anchored to Bailey panels buried in the ground. A ½-in. diameter wire rope for towing was linked to this cable through a snatch block. During operations of the ferry, the main cable used to be greased once a month by a person standing on a truck on board the raft and the pulley used to be greased more frequently.

4. **Bridge Components and other Stores actually used at the Kaljani site are listed below:**

Panels	42 No.	(2 No. extra for anchoring ends of cable)
Transoms	24 No.	
Stringer, plain	24 No.	
Bracing frame	36 No.	
Sway braces	16 No.	
Rope SWR, 3 in.	1,000 ft	
do 1½ in.	500 ft	
Bulldog clips	20 No.	
Double Block	1 No.	(To tighten the suspended cable.)
Snatch Block	1 No.	

Pins, clamp, bolts, etc. as required.

APPENDIX 3

Details for the Ferry on the Sil-Torsa at Silbariaghat

1. **Cost of Operations**

Running charges (10 to 12 hours daily)

(a) **Boat Crew**

4 O. B. Engine drivers at Rs 110 per month	Rs	440
2 Head Boatmen at Rs 65 per month		130
8 Boatmen at Rs 50 per month		400
Total	Rs	970 per month.

4 extra boatmen for the flood season at Rs 50 per month, for 5 months, Rs 200 per month

Work Establishment charges yearly Rs 12,690 or 12,500

(b) 22/25 H.P. Outboard engine 5 No. per year at Rs 3,500 (fully used up)	Rs	17,500
Overhaul, repairs and spare parts 6 No. × 1,500	Rs	9,000
(c) Cost of Petrol & Lubricants,—Flood season 16 to 24 gallon petrol and 1 to 1½ gallon lubricating oil. Special grease for under-water use, 1 lb per day. Dry season—Petrol 8 gallons and lubricating oil ½ gallon. Cost for one year	Rs	18,000
(d) Average for repairs to boats	Rs	3,000
Sundry stores and carriages	Rs	2,000
Total running charges	Rs	62,000

2. **Approaches, Bridges and Ghats**

(a) Work Assistant to supervise 1 at 100/ = 100; Mate 1 at 58/ = 58; Gang Mazdoor 12 at 50 = 600 } or Rs 758 per month.

Yearly expenditure on work-charged Estt. Rs 10,000

(b) Temporary bridges and ghats

Contract works and value of materials consumed = Rs 10,000

Total = Rs 20,000

Total expenditure Rs 95,000 including contingencies and agency charges.

3. Data on Traffic

1. Daily average vehicular traffic

January to March	90 vehicles	Bullock cart traffic is very small
April to June	125 ,,	
July to September	65 ,,	
October to December	85 ,,	

(Buses 30 per cent; Trucks 50 per cent; Passenger vehicles 20 per cent.)

4. Particulars of the River

1. Dry season: Width of the river 150 to 200 ft by the best crossing.
   Minimum depth at ferry site:
   For country boats 3 ft
   For aluminium pontoons 2 ft 6 in.
   Velocity of current 3 to 4 ft per second
   Rise and fall in 24 hours ... Small
2. Wet season: Width of the crossing 1,000 ft maximum
   600 ft minimum
   Rise and fall in 24 hours 2 ft to 2 ft 6 in.
   Velocity of flow — 8 to 9 ft per sec.
   Stoppage of ferries—3 to 4 times per year lasting 1 to 3 days
   (Provided O. B. Engines are available.)

5. Particulars on Power Units

1. O. B. Engines used or in service in last 5 years:
   16 H. P. engines 3
   22 H. P. engines 22, and
   25 H. P. engines 5 (Recent supplies)

New model 25 H. P. machines have light metal propeller blades which wear away quickly in contact with sandy beds. In earlier models, the blades which were of brass were more durable.

Parts of the engine which get worn out or damaged are gaskets, propeller blades and pin, powerhead, plug, gear pinion, big and small bearings, propeller shaft and bush, piston and rings, etc.

INFORMATION SECTION

CEMENT GROUTED ROADS

By

E. A. NADIRSHAH

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SYNOPSIS

An account of some of the experimental lengths of cement grouted roads recently (1950-54) constructed is given. Details of the behaviour of the experimental lengths and costs are included.

1. INTRODUCTION

The Paper Grouted Concrete Roads* presented by the Author in 1951 gave details of the method of construction and costs of some of the experimental stretches as compared to concrete roads constructed by the conventional premix method. The discussion showed that considerable scepticism about the success of this type of construction existed in view of the limited experience and data that were available. Since then 19 stretches of grouted roads of varying lenghts were constructed in different States. A very large portion of the work was carried out in Madhya Pradesh with success. The specification and the method of construction adopted for these have been very much the same as those given in the Paper and only in one case a separate premixed concrete wearing course had been provided over the grouted surface. In a few cases the work undertaken has been

*Published in the Journal of the Indian Roads Congress, Vol. XVI-1, November, 1951.

quite small and, therefore, the advantages of higher speed of construction, reduced cost in handling of materials, etc., have not been clearly reflected in the costs. However, some of the large-scale grouted constructions carried out have very clearly brought out the savings in material, labour, and overall cost as compared to the conventional premix method. The results of these experiments should by no means be considered as conclusive and suggest the need for further experimental work on a scientific basis in a laboratory as well as in the field.

This note is published to supplement the data in the Paper and stimulate further thought. The data include information collected on the new constructions carried out.

2. SOME RECENT EXPERIMENTS

2.1. Details of the experimental stretches newly laid are furnished in Appendix 1. The general specifications adopted on these constructions did not vary appreciably from each other. Only in one case, due to doubtful wearing qualities of local aggregates, a two-course construction was adopted using grouted method for the bottom and premix method for the top course.

2.2. Aggregates

Size. The size of coarse aggregates ranged from a minimum of 1 in. to a maximum of 3 in. In most cases, sizes from 1 to $2\frac{1}{2}$ in. were used. For fine aggregates, sand passing $\frac{1}{8}$ in. sieve was used wherever available.

Placing. The aggregate was first stacked at the side of the road and then placed between the side forms manually. This involved double handling of major portion of the materials. This extra handling resulted in extra labour cost and did not prove much cheaper than labour for premix pavements. Besides, a certain amount of delay involved in manual placement of aggregates hindered the speed of operations, indirectly adding to the cost. The most desirable method would be to prepare the subgrade and keep the side forms fixed for sufficient lengths so that aggregates could be dumped directly on the subgrade from trucks, preventing the possibility of loose earth getting mixed up with the aggregates as with stacking done at the side.

Rolling. In all cases, the aggregates were consolidated by rolling and in one instance, by a vibrating tamper before they

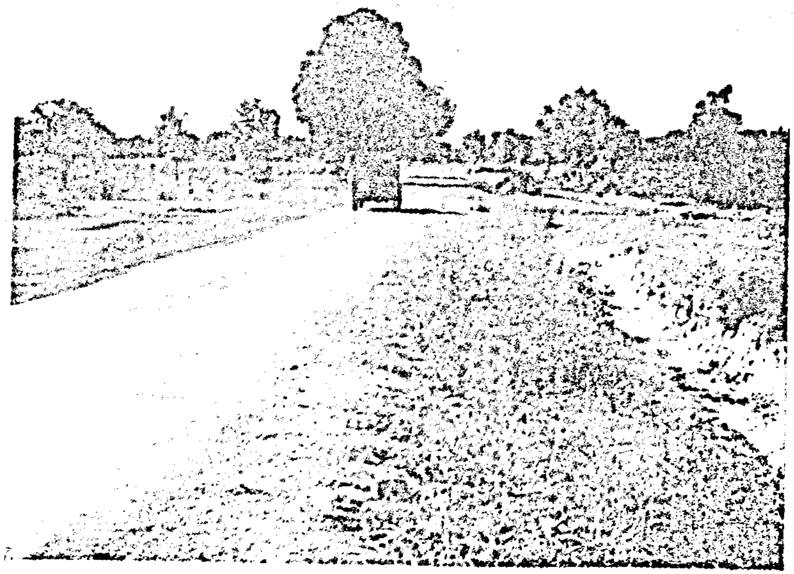

Photo 1.
Kotma-Bhalumara Road, Finished Grouted Concrete Road

Photo 2.
Grouted Concrete Slab in Patna City—Marufganj Road

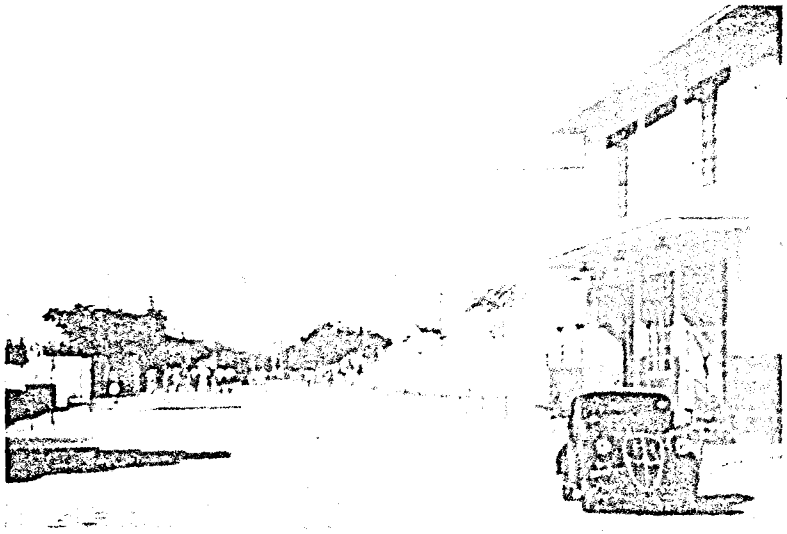

Photo 3. **Approach Road to Hindustan Aircraft Ltd.**
Left half 15 ft width finished Premixed Concrete Road and right half 15 ft width Cement Grouted Road

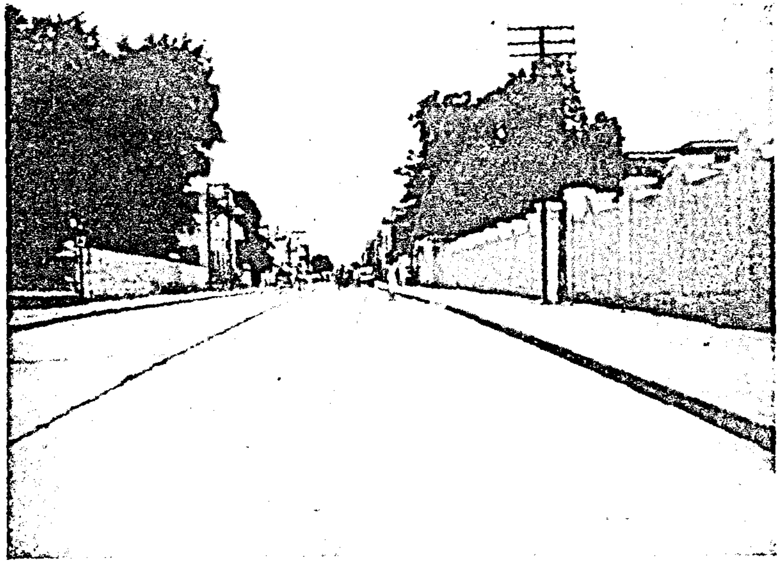

Photo 4. **Grouted Concrete Road—Miss Manning's Girls' School to Post Office, Hyderabad**

were grouted. This was done with a view to minimise the voids and conserve cement. Eight and ten ton rollers were used for rolling in 6 cases. For others, 6-ton tandem rollers, hand rollers or vibrating hand tampers were used.

The optimum extent of the rolling was judged by the degree of compaction of the coarse aggregates spread on the subgrade. To obtain a finished slab thickness of 4 in., $4\frac{1}{2}$ in. of coarse aggregates is spread on the subgrade prior to rolling. For a 5 in. finished slab, a layer $\frac{5}{8}$–$\frac{3}{4}$ in. in excess of the required finished thickness was provided.

2.3. Cement Grouting

Proportions of Grout. The proportions for the cement grout varied from 1 part of cement to $1\frac{1}{2}$-2 parts of sand. The grout with higher percentage of cement showed slightly better penetration than the one of lower cement content. Volumetric proportioning of sand and cement were adopted in all the experiments.

Final Proportions. The resulting proportions of concrete as finished, based on the actual consumption of aggregates, ranged from 1 : 1.5 : 6 to 1 : 2.4 : 8.3. The variation was due to the proportions of the grout, the thicknesses of the aggregate layers and percentages of voids which varied with the degree of campaction on each job. The percentage savings in cement consumption varied from 8.5 per cent to as much as 31 per cent, depending upon factors like the grading of the coarse aggregates, the grout proportions, etc.

Wetting Agent. $\frac{1}{4}$ pint of the proprietary wetting agent per cwt of cement was used as recommended by the manufacturers. The increase in the fluidity of the grout with the addition of wetting agents was no doubt very marked and helped in maintaining fairly low water-cement ratio for satisfactory penetration

Water Content. The water contents of the grout for the successful penetration yielded water-cement ratios varying from 0.58 to 0.75 by weight. In one experiment the water-cement ratio of 0.65 by weight was adopted for grouting the lower portions of the aggregates and for the upper layers, a grout of lower fluidity with a water-cement ratio of .55 was used. Where the final proportions of concrete worked out to be quite lean, due to smaller void spaces or the use of a leaner grout proportion, water-cement ratio had to be higher for satisfactory penetration being in the neighbourhood of 0.70

Method of Compaction. Compaction was by hand tamping, but in one case a vibrating hand screed was used. The pavement was not rolled after the pouring of the grout in any of the experiments.

2.4. Surface Finish

After grouting the surfaces were checked by a template and wherever depressions were noticed additional small sized ballast was sprinkled and extra grout poured to make up an even surface.

Special Wearing Course. In all the works except one, the surface was finished by tamping small sized chippings below $\frac{3}{4}$ in. size into the surface of the pavement after the grouting was completed.

On Mahatma Gandhi Road at Chanda in Madhya Pradesh, a two-course construction had to be resorted to due to the doubtful wearing qualities of the locally available aggregates. The top 1–$1\frac{1}{2}$ in. thickness was, therefore, finished with premixed concrete of 1 : 1.75 : 3.5 proportion using coarse aggregates with superior wearing qualities. In spite of the extra labour involved in using two different specifications on the same job, the advantage of grouted road technique is reflected in the actual saving of labour and cement consumption.

2.5. Expansion and Dummy Joints

Expansion joint spacings varied from 33 to 140 ft whereas dummy joint spacings were from 15 to 30 ft. It appears that a closer spacing of dummy joints (15 ft centres) is desirable to localise shrinkage or contraction cracks in road slabs.

3. PERFORMANCE

Observations made during periodical inspections are given below :

3.1. Cracking

These roads exhibited a slightly greater tendency towards transverse and corner cracking than premixed roads.

In one instance, the shrinkage cracks on the grout surface caused by excessive drying shrinkage were noticed where the water-cement ratio was higher than 0.6. This, however, was obviated by using a thinner grout for the grouting of the lower portions followed by a grout of thicker consistency for the remaining depth. On constructions where the proportion of the grout used was leaner and water-cement ratios were as high as 0.70 to 0.75, no such tendency to shrinkage cracking was however observed.

3.2. Pitting of the Surface

In a few cases, the surface finish of the slabs showed signs of pitting. This was attributed to the fact that in these cases the grout before spreading on the surface was not screened through a mesh nor any attempts made to break up the lumps of sand formed in the grout.

Barring the above the average performance of the road constructed by cement grouted method has been quite satisfactory. See Appendix 1.

4. CONCLUSIONS

4.1. General

From the experimental cement grouted roads so far done sufficient data truly representing the merits and demerits of this type of construction have not been obtained. Besides, almost all the specifications adopted for the experiments so far were more or less identical and alternative methods have not been explored. In smaller sized jobs, it has not been possible to assess correctly the economy of this type of construction. More favourable results might be obtained in a bigger job, by the adoption of improved methods of construction, such as the use of a grout pump, programmed continuous operations, viz., spreading of coarse aggregate, rolling, grouting and finishing.

4.2. Suggestions for Further Experimental Work

In emphasising the need of immediate study of variables in the process at the laboratory and by a large number of experiments under different traffic and subgrade conditions in field, the following suggestions are offered :

The experimental work should include :

(i) Determination of the efficacy of the wetting agent for increasing the fluidity or the penetration of the grout.

(ii) The effects of the addition of such dispersing agents on shrinkage and binding characteristics of the mortar.

(iii) The effects on performance of the pavement by (a) rolling of aggregates only before grouting and (b) rolling both before and after grouting should be investigated under identical conditions. It is quite likely that light rolling of aggregates after penetration of the grout will remove any weak spots in the loosely bonded structure of the aggregate, resulting in a higher density and properly interlocked aggregates having higher resistance to shrinkage cracks and improved wearing qualities.

(iv) Degree of grout penetration with different degrees of compaction.

Attempt should be made to evolve a suitable specification for grouted or composite road construction, which would have strength and properties comparable to 1 : 2 : 4 premix concrete pavements.

Besides savings in material and labour, factors such as speed of construction and their effect on the outturn should be reckoned in judging the suitability of this type of construction.

RESEARCH SECTION

1. USE OF LINSEED OIL-CAKE AS A STABILIZER IN EARTH CONSTRUCTION
2. LABORATORY EXPERIMENTS IN SAND STABILIZATION

USE OF LINSEED OIL-CAKE AS A STABILIZER IN EARTH CONSTRUCTION

By

MANOHAR LAL, M. Sc.

AND

B. L. DHAWAN, M. Sc.

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SYNOPSIS

Laboratory experiments on the use of linseed oil-cake as a stabilizer for use in plastering are described. These include wetting and drying, compressive and tensile tests. It is concluded that mud plasters with linseed oil-cake can be reused without loss of strength.

1. INTRODUCTION

In a previous Research publication of the Indian Roads Congress[1] it was shown that linseed oil-cake acts as a water-proofing agent for alkaline soils (P. R. A. Classification, A-4). This property of the cake has now been utilized for making non-erodable plaster for stabilized soil walls.

Also emulsions or water-insoluble binders like cement cannot be used again with water to give the same results after mechanical breakdown of the soil. Puri[2] has reported that high binding values obtained with soluble binders like molasses can be restored again with water alone after the mechanical breakdown of the soil. It is, therefore, of interest to study how linseed oil cake, an insoluble binder, will affect the stability of the soil when mixed with it.

2. THE SOIL

The soil used for the experiments is silty clay more than 35 per cent of which passed through 200 sieve and belonged to group A-4 (P. R. A. Classification). The sample was dried, powdered and passed through a 2 mm. sieve before the various tests were carried out.

3. THE STABILIZING AGENT

A fresh sample of the linseed oil cake used as stabilizing agent was obtained from the local market and was found to have the following composition :

	Per cent
Moisture	10.8
Oil	7.9
Albuminoids	30.7
Carbohydrates	36.2
Fibre	9.6
Mineral matter	4.8
	100.0

4. WETTING AND DRYING TESTS

4.1. The purpose of this test was to determine the losses produced by repeated wetting and drying of compacted specimens of soil-linseed cake mixture of known composition which gave an idea of the stability of the soil-linseed cake mixture when subjected to saturation by heavy rainfall and drying by sun.

4.2. Tests were carried out by making specimens in a split cylindrical mould which had a known internal volume (3 in. long and 2 in. diameter). A pre-determined weight of the dry soil mixed with different percentages of linseed cake was compacted at optimum moisture by placing the soil in the mould between the top and bottom plungers which were then pressed to the same weight in the soil compaction machine.

4.3. The compacted specimens were cured in a moist closet having 90-95 per cent humidity. These were dried to constant height in an oven at 50-55 degrees centigrade and then removed, cooled and immersed in water for five hours at room temperature. Thereafter they were taken out of water and dried in an oven at

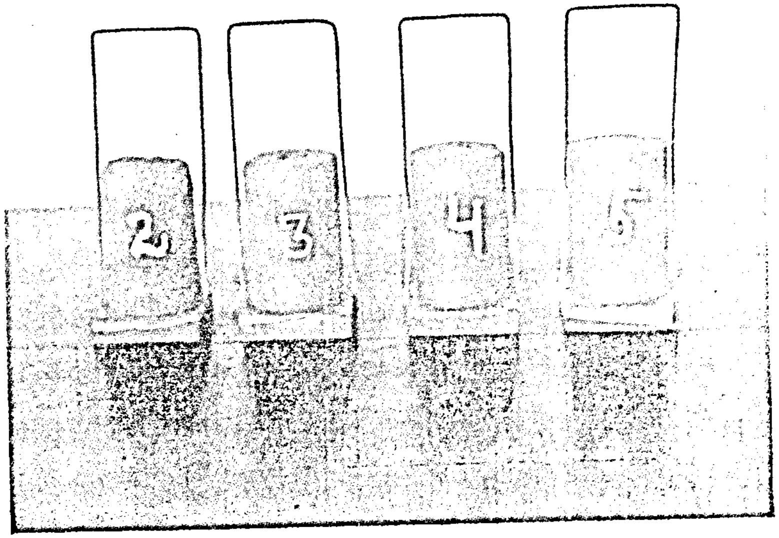

Photo 1. Soil blocks containing 2 to 5 per cent linseed oil cake after 12 cycles of wetting and drying

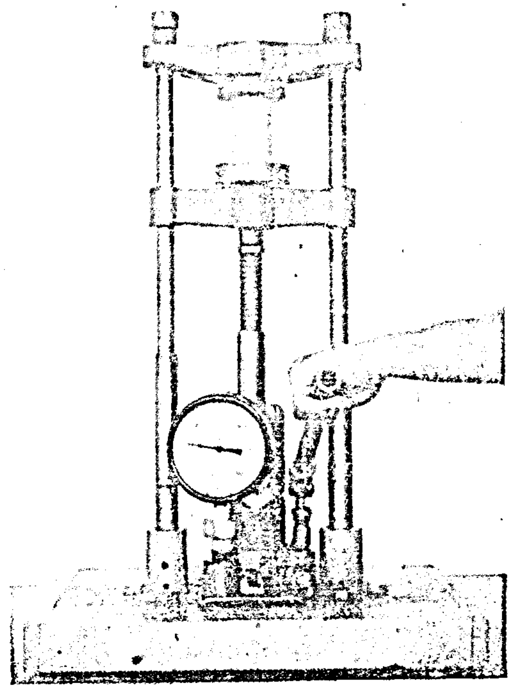

Photo 2. Compression Machine

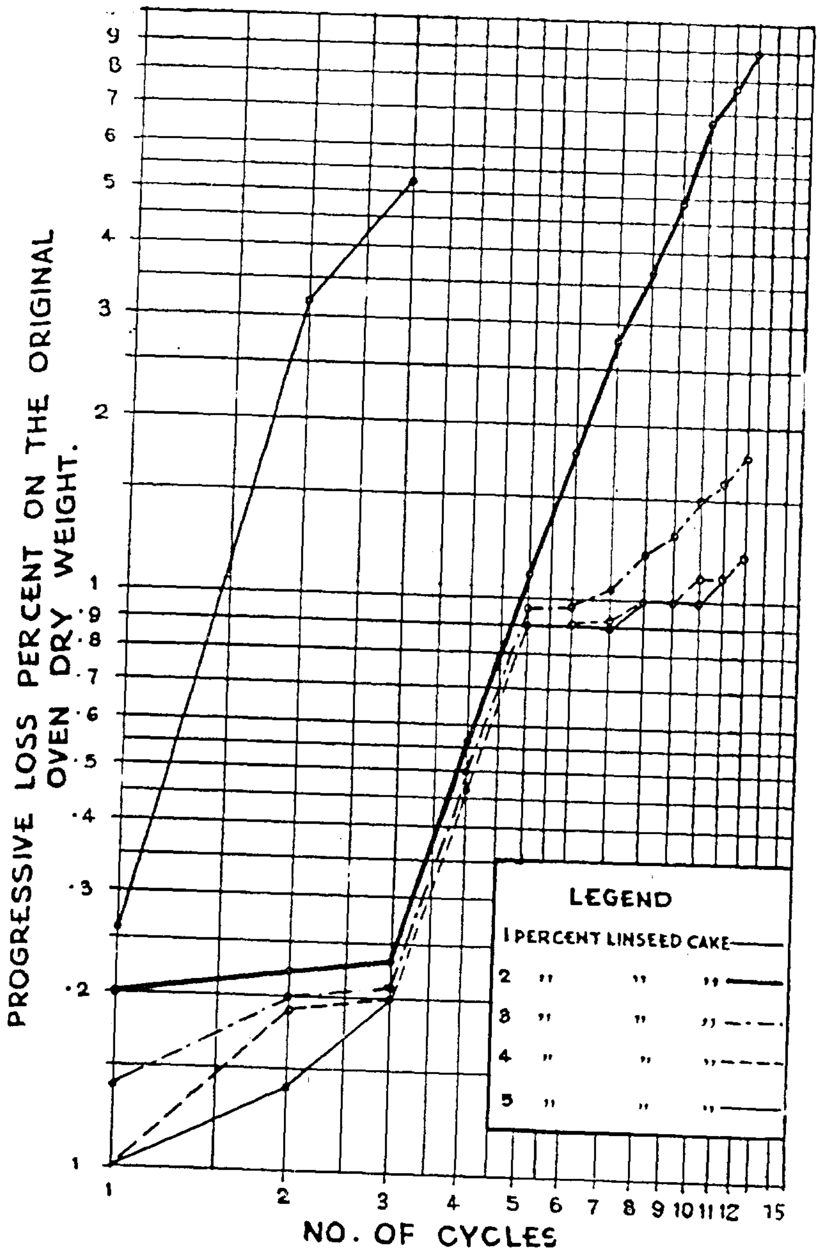

Fig. 1. Loss in weight of Linseed Oil Cake soil mixtures when subject to 12 cycles of wetting and drying.

50 to 55 degrees centigrade for 42 hours. After this the specimens were removed from the oven, cooled and the loose material on the surface was removed by brushing. The specimens were then weighed to calculate the percentage loss. The specimens were subjected to 12 cycles as described above.

This procedure was similar to the standard A. S. T. M. [3] wetting and drying test for compacted soil cement mixtures, except that a modification in temperature for drying was made to avoid charring of the cake at higher temperature. The results obtained with different percentages of linseed cake are given in Table 1 and shown in Fig. 1. The condition of the blocks after 12 cycles of wetting and drying is shown in Photo 1. The numbers depicted in the Photo indicate the percentage of the linseed oil-cake used. Block numbered 2 shows some disintegration whereas No. 3, 4 and 5 are intact.

TABLE 1

Loss in Weight of Linseed Oil Cake-Soil Mixtures during Wetting and Drying

Description	Percentage Loss in Weight during Wetting & Drying											
	Number of Cycles											
	1	2	3	4	5	6	7	8	9	10	11	12
Soil (Blank)	←——Disintegrated immediately——→											
Soil+1 per cent linseed cake	0.26	3.20	5.20	←——Disintegrated——→								
Soil+2 per cent linseed cake	0.20	0.22	0.23	0.56	1.10	1.80	2.80	3.70	4.90	6.80	7.80	9.00
Soil+3 per cent linseed cake	0.14	0.20	0.21	0.50	0.97	0.98	1.05	1.20	1.30	1.50	1.60	1.80
Soil+4 per cent linseed cake	0.10	0.19	0.20	0.47	0.90	0.91	0.92	1.00	1.00	1.10	1.10	1.20
Soil+5 per cent linseed cake	0.10	0.14	0.20	0.47	0.90	0.90	0.90	1.00	1.00	1.00	1.10	1.20

It is seen from Table 1 that when soil is mixed with 3 per cent or more of linseed cake, the loss at the end of 12 cycles is low.

This indicates satisfactory hardness and durability. Moreover, the blocks did not disintegrate even after 12 cycles of wetting and drying. It is, therefore, concluded that the linseed oil cake acts as a binder for soil particles.

5. TESTS FOR COMPRESSIVE AND TENSILE STRENGTHS

5.1. Blocks and briquettes of soil mixed with linseed cake were broken and remade and strength was recorded each time. Prior to moulding the blocks and briquettes, moisture density tests were conducted on representative mixtures (containing different percentages of linseed cake) in accordance with A. S. T. M. method D 558-40T. Test specimens were moulded at optimum moisture content and maximum density. In remoulding the blocks and briquettes, the same quantity of soil and water were taken in each particular case, the same moulds were used and the top and bottom plungers were pressed to the same height in the compaction machine in order to maintain similar conditions of compaction. All the specimens were cured in a moist closet at 90-95 per cent humidity and tested after drying to constant weight in the oven at 50-55 degrees centigrade.

5.2. Compressive strength tests were made on specimens, 2 in. diameter and 3 in. in height, using the compression machine shown in Photo 2. The tensile strength tests were made on standard size briquettes. The results are given in Tables 2 and 3.

TABLE 2

Compressive Strength of Soil Stabilized With Linseed Oil Cake as affected by Breaking and Remaking

Sl. No.	Description	Compressive strength in lb per sq. in. (average of 4 blocks)	Compressive strength after breaking and remaking in lb per sq. in. (average of 4 blocks)
1.	Soil (blank)	286	284
2.	Soil & 1 per cent linseed cake	305	306
3.	Soil & 2 ,, ,, ,, ,,	347	347
4.	Soil & 3 ,, ,, ,, ,,	386	375
5.	Soil & 4 ,, ,, ,, ,,	439	435
6.	Soil & 5 ,, ,, ,, ,,	449	449

TABLE 3

Tensile Strength of Soil Stabilized with Linseed Oil Cake as affected by Breaking and Remaking

Sl. No.	Description	Tensile strength in lb per sq. in. (average of 4 briquettes)	Tensile strength after breaking and remaking in lb per sq. in. (average of 4 briquettes)
1.	Soil (blank)	53	53
2.	Soil & 1 per cent linseed cake	62	64
3.	Soil & 2 ,, ,, ,, ,,	64	70
4.	Soil & 3 ,, ,, ,, ,,	72	75
5.	Soil & 4 ,, ,, ,, ,,	79	80
6.	Soil & 5 ,, ,, ,, ,,	92	92

From the above Tables it is clear that the increased strength with cake is maintained when blocks and briquettes are broken and remade.

6. NON-ERODABLE PLASTER

6.1. The waterproofing and binding properties of the linseed cake suggested its use for making a non-erodable plaster. Narayanamurti and Dagg[4] have also developed cheap and satisfactory adhesives from castor cake and its proteins. In the present investigation, 3 per cent linseed cake was mixed with soil to produce an adhesive plaster which could stick well to stabilized soil surfaces. This percentage of cake was chosen as it was the minimum requried to impart the requisite properties for practical use.

6.2. Soil-cement blocks were made by compacting 200 grams of cement-soil mixture at optimum moisture to a density of 1.9 to 2.0. These were cured for seven days in a moist closet at 90-95 per cent humidity and then allowed to dry in shade. Two such blocks were joined together by ½ in. thick mud plasters of different mixtures given in Table 4. The mud plaster was prepared by mixing the soil, linseed cake and '*bhusa*', as the case may be, with water and was left over for a week, working it up every day and adding sufficient water to make up the loss due to evaporation. The soil got dispersed, straw and cake thoroughly rotted during the process.

6.3. The blocks thus joined were cured in the humidity chamber for three days and then dried in the shade before finally drying in the oven at 50–55 degrees centigrade. The specimens were then subjected to the adhesion test described by Mehra and his co-workers [5]. The pull per unit area was calculated and the results are given in Table 4.

6.4. In order to study the adhesion of plaster to the stabliliz-ed soil walls containing no cement, compacted loam soil blocks were made. Two such blocks were joined together by ½ in. thick plaster prepared as described in para. 6.2. The blocks so joined were kept in the humidity chamber at 90–95 per cent humidity for three days and were then dried in the shade before finally drying in the oven at 50–55 degrees centigrade. These were then sub jected to adhesion test and the results which indicate that the linseed cake soil plaster can stick well to soil walls are given in Table 4.

TABLE 4

Adhesion of Plaster when applied to Compacted Blocks

Sl. No.	Plaster used	Adhesive strength in lb per sq. in. using soil-cement blocks (average of 6 tests)	Adhesive strength in lb per sq. in. using soil blocks (average of 6 tests)
1.	Soil only and kept moist for a week	Block gave way while fitting	Blocks gave way while fitting
2.	Soil & 2 per cent *bhusa* kept moist for a week	2.85	2.14
3.	Soil & 3 per cent linseed cake and 2 per cent *bhusa* kept moist for a week	9.9	8.2

It is, therefore, clear that the adhesive capacity of the soil increased considerably when mixed with linseed cake. This increased adhesion is probably due to the presence of proteins in the linseed cake.

6.5. In order to study if the adhesiveness of the plaster was retained after mechanical breakdown, the used plaster was

scratched from blocks after four months and was thoroughly mixed with water. It was applied to the soil cement blocks and adhesion was tested again. The results are shown in Table 5.

6.6. The possibility of deterioration of the bond imparted by soil-cake plaster with repeated changes in humidity was also studied. Soil cement blocks joined with the plaster were subjected to changes in humidity by storing them alternately in vacuum desicators where 95 and 10 per cent humidities were maintained with the help of sulphuric acid mixtures. The blocks were subjec-ted to twelve such cycles of humidity changes and were finally dried at 50–55 degrees centigrade and then subjected to adhesion test. The results are given in Table 5 which show that humidity changes do not alter the adhesive properties.

TABLE 5

Strength of Soil-Linseed Cake Plaster as affected by Breaking and Remaking; Humidity Changes

Sl. No.	Description of plaster	Adhesive strength in lb per sq. in.		
		When original plaster was used	After breaking and remaking plaster	After humidity changes
1.	Soil, 3 per cent linseed cake and 2 per cent *bhusa*	9.2	11.0	9.2
2.	Soil, 3 per cent linseed cake & 2 per cent *bhusa*	10.4	9.7	8.2
3.	Soil, 3 per cent linseed cake & 2 per cent *bhusa*	8.1	9.4	10.0
4.	Soil, 3 per cent linseed cake & 2 per cent *bhusa*	12.2	8.1	8.5
5.	Soil, 3 per cent linseed cake & 2 per cent *bhusa*	9.6	10.2	8.1
6.	Soil, 3 per cent linseed cake & 2 per cent *bhusa*	9.7	9.2	10.2

The adhesive strength of the soil-linseed cake plaster is, therefore, maintained after breaking and remaking the plaster.

7. ADDITION OF INSECTICIDES

7.1. While the incorporation of linseed cake with soil increased the adhesive capacity and strength of the soil it was found that it was susceptible to attack by white ants. A search for insecticides was then made and it was found that the addition of 0.4 per cent of creosote oil was effective. Tests using the plaster on a wall gave promising results. Further work in this direction is in progress.

8. CONCLUSIONS

8.1. Wetting and drying tests showed that the soil mixed with 3 per cent or more of linseed cake developed good binding properties.

8.2. Compressive and tensile strengths of the soil increased when mixed with cake and the increased strength remained practically unchanged when broken and remade.

8.3. Linseed cake-soil plaster stuck well to the stabilized soil surfaces and after mechanical breakdown the same plaster could be used with water without altering the adhesive forces.

8.4. Humidity changes did not affect the adhesive property of imparted by the soil-cake plaster.

ACKNOWLEDGEMENT

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LABORATORY EXPERIMENTS IN SAND STABILIZATION

By

H. L. UPPAL

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SYNOPSIS

Laboratory experiments on the compressive strength and resistance to the softening action of water of lime-sand-molasses mixes in various proportions are reported. The effect on the compressive strength of washing the blocks of the mixes with water and water saturated with CO_2 is described.

The chemistry of the reaction that takes place when lime is mixed with molasses is discussed.

1. INTRODUCTION

1.1. In areas where sand is the only available material for construction of roads the maximum use of the material is necessary for economy. But sand has no cohesion in the dry state and must be made to attain a fair amount of compressive strength by the use of additives before it can be used.

1.2. Some investigations on the stabilization of sand have already been carried out in England. The additives tried were cement, lime, vinsol* (a resin), lomix (a bitumen emulsion), S. S. O. (bituminous oils), and S. R. O. waxy, and X. R. O. (a bituminous oil).

1.3. Promising results have been claimed with cement, lime and lomix, but apart from the question of importing them, the concentrations required are so high that they may be considered uneconomical for use in India.

2. OBJECT OF THE STUDY

2.1. The object of the study reported here, therefore, was to stabilize a fine sand with some of the cheap additives available in India. The additives considered most suitable for the purpose were (i) lime and (ii) molasses.

For judging the suitability of a stabilized soil for road construction the tests recognised are (i) compressive strengh on the laboratory blocks, and (ii) resistance to the softening action of water. The limiting values for these were arbitrarily fixed as :

(i) Compressive strength not less than 250 lb per sq. in. (though a compressive strength of 400 lb per sq. in. has been suggested for base courses where heavy military traffic is likely to come up).

(ii) Resistance to softening action of water. No loss of material in the first 24 hours of immersion.

3. SAND USED FOR THE STUDY

3.1. The sand used was from Jumna river. It was mostly fine sand with 0.61 per cent of white mica. The mechanical analysis of the sample is given in Table 1 and graphically shown in Fig. 1.

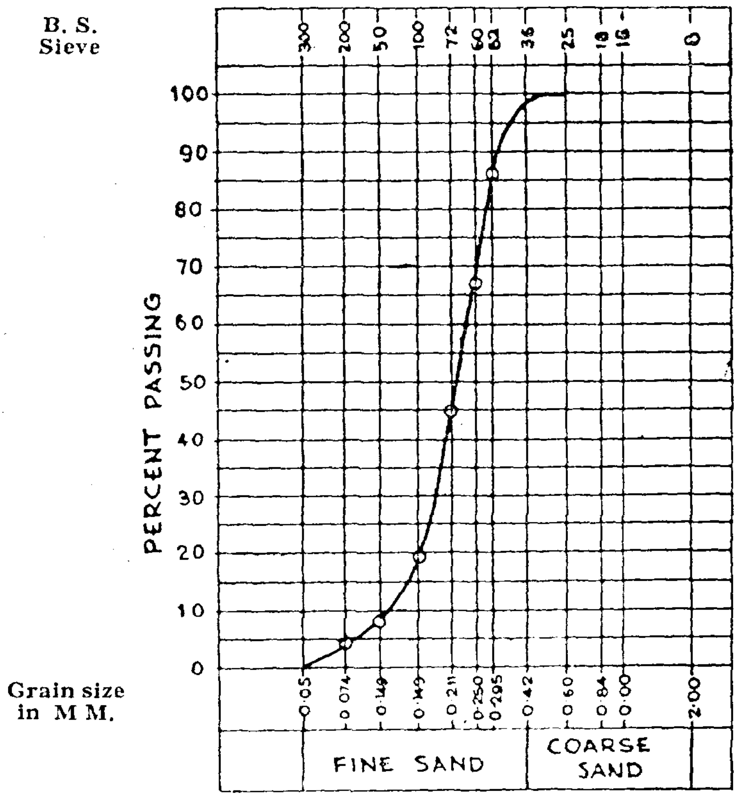

Fig. 1. Grain Size Distribution Curve for Jumna Sand.

TABLE 1

Mechanical Analysis of the Jumna Sand

B. S. Sieves	Percentage passing
No. 25	100. 0
36	99.10
52	85.34
60	66.80
72	44.84
100	18.92
150	7.50
200	4.70
300	Nil

4. EFFECT OF LIME

4.1. The effect of lime on the compressive strength of the sand was determined by making blocks in the modified Abbots Cylinder[2] with increasing percentage of lime.

4.2. **Optimum Moisture of Lime-Sand Mixtures.** For a desired density it is necessary to have the correct moisture which varies with the proportion of lime in the mixture. Therefore, the optimum moisture was determined with two extreme proportions viz., 2.5 and 10 per cent using Abbots Cylinder and 15 blows. The optimum moisture was found to be 10 per cent in both the cases. Therefore 10 per cent was taken to be the optimum moisture for other concentrations also.

4.3. Blocks with 200 gm of lime-sand mixtures were compacted using Abbots Cylinder. The blocks had a diameter of 2.5 in. and a height of 1.5 in. with dry bulk densities varying between 1.5 and 1.55. These were cured under wet sand for a week and then dried to constant weight in an incubator at 35° C before subjecting them to the compression. The results obtained are given in Table 2 and shown in Fig. 2.

TABLE 2

Compressive Strength of Lime-Sand Mixtures

Percentage of lime	Compressive strength* lb per sq. in.
0	Uncompactable
1	-do-
2.5	5.8
5.0	41.74
7.5	89.52
10.0	148.4

4.4. From the results in Table 2 it appears that compressive strength of lime-sand mixtures is very low even when 10

* Averages of 3 tests.

per cent lime is used. Higher proportions were not tried as they were considered unecomomical.

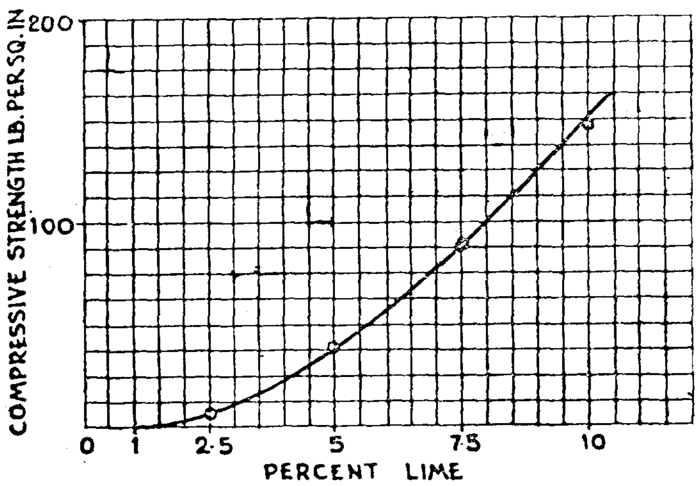

Fig. 2. Compressive Strength of Lime-Sand Mixtures.

5. USE OF MOLASSES

5.1. Molasses which is a by-product from sugar factories is now mainly used for the manufacture of alcohol but in areas close to the factories it is available cheap.

5.2. Because molasses possesses binding properties attempts had been made by various workers [3-14] to use it as a surfacing material for roads in place of bitumen. A number of trials using molasses, or molasses mixed with lime or after resinification were made but the resultant product was found to be very much inferior to bitumen, and consequently could not stand trials in the field.

5.3. Some of the laboratory experiments[15] have shown that soil can be successfully stabilised with molasses. No attempt has, however, been made to stabilize sand with this material.

5.4. The molasses used for the study were got from the local market after ascertaining that it was fairly fresh. The analysis of the molasses used is given in Table 3.

TABLE 3

Analysis of Molasses Used in the Study

	Per cent by weight
Sucrose (determined by saccharimeter)	31.25
Brix (Total solids)	86.60
Reducing sugars (determined volumetrically)	21.70
Gummy matter	1.38
Ash (Inorganic matter)	11.40
Organic matter (Non-sugars)	11.91
Moisture	22.36

5.5. Compaction of Lime-Sand Molasses Blocks

Blocks for testing the compressive strength were made by first mixing pure lime (Calcium hydride) with sand and then required amounts of molasses were added to make up the optimum moisture content and the whole mixed. Since the moisture of compaction has considerable influence on the resultant density of the mixture, and consequently the compressive strength, to start with, optimum moistures for 5 and 10 per cent lime-sand mixtures were determined with increasing percentage of molasses i.e., 2.5, 5.0, 7.5 and 10 per cent. The optimum moisture for these mixes are given in Table 4.

TABLE 4

Optimum Moisture for Sand-Lime Mixtures With increasing Percentages of Molasses

Percentage of lime in the mixture	Optimum Moisture for Percentage of molasses			
	2.5	5.0	7.5	10
5	11.5	11.7	11.4	11.4
10	12.0	11.0	11.0	11.0

It was found during the test that a moisture content a little less than the optimum is more convenient to work with, as it avoided the sticking of blocks to the compacting unit.

Moisture content of 11 per cent was, therefore, used for all lime-sand mixtures when compacted with molasses up to 7.5 per cent. For the 10 per cent mix the most convenient moisture content was found to be 10 per cent. This small deviation from the optimum moisture content did not make any appreciable change in the dry bulk density. The tests were therefore carried out by taking 2.5, 5, 7.5 and 10 per cent lime-sand mixtures and compacting them at 15 blows in the Abbots Compaction Apparatus with 2, 4, 6, 8 & 10 per cent molasses.

5.6. Compressive Strength of Compacted Block

The compacted blocks were cured and dried as described in para. 4.3 and then subjected to compression. The results are given in Table 5 and shown in Fig. 3. It will be seen that compressive strength increases with the increase in the proportions of lime and molasses. However, when a mixture with small percentage of lime is compacted with a high percentage of molas-

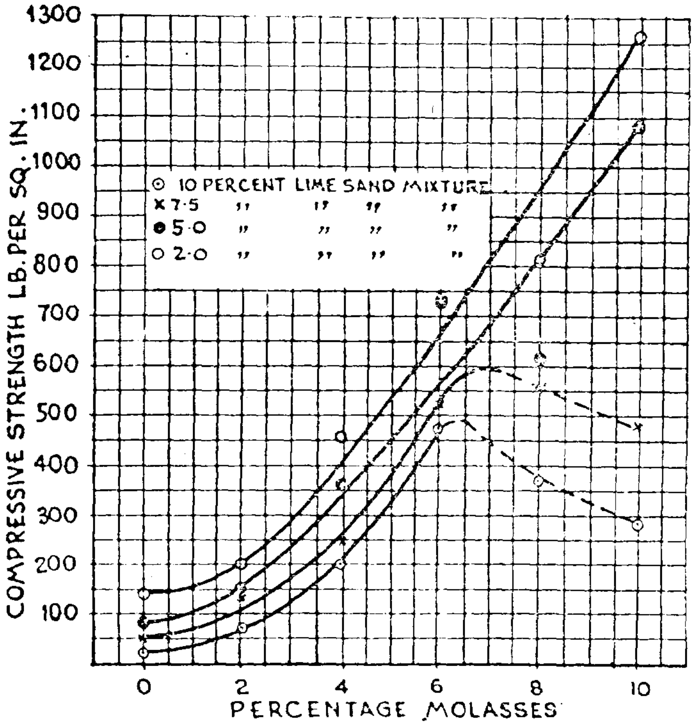

Fig. 3. Compressive Strength of Compacted Lime-Sand-Molasses Mixtures

ses, cracks developed in the blocks, resulting in reduced compressive strength. The reduction may partly be due to higher percentage of moisture retained by molasses, when the percentage of lime is small. A compressive strength of 250 lb per sq. in. can be obtained only with a minimum of 2.5 per cent lime-sand mixture when compacted with at least 4.5 per cent molasses or 5 per cent lime-sand mixer with a minimum of 4 per cent molasses by weight. For higher compressive strength the required concentrations of lime and molasses can be selected from the tables.

6. COMPRESSIVE STRENGTH OF BLOCKS AFTER WASHING WITH WATER

6.1. The above results show that lime-sand mixture attain a fairly good strength when compacted with molasses and the data may therefore be useful in arid regions. It is, however, not known how the compressive strength of these blocks will be affected when they remain in contact with water for some time. In order to know the compressive strength of the blocks when washed with water another set of blocks using proportion of lime and molasses given in Table 5 was prepared and immersed under water, flowing at the rate of about one cu. ft. per hour. The blocks were taken out after 3 days, dried in the incubator to constant weight and then subjected to compression. The results are given in Table 6 and shown in Fig. 4.

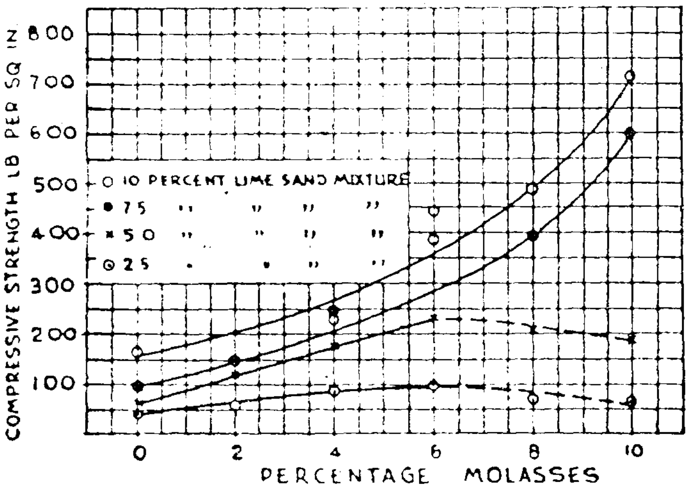

Fig. 4. Effect of Washing on the Compressive Strength of Blocks made with Lime-Sand and Molasses.

TABLE 5

Compressive Strength of Lime-Sand Mixtures with Increasing Percentages of Molasses

	Per cent of Molasses											
	0		2		4		6		8		10	
Per-cent lime in sand	Dry bulk density	Compressive strength lb per sq. in.	Dry bulk density	Compressive strength lb per sq. in	Dry bulk density	Compressive strength lb per sq. in.	Dry bulk density	Compressive strength lb per sq. in.	Dry bulk density	Compressive strength lb per sq. in.	Dry bulk density	Compressive strength lb per sq. in.
2.5	1.56	20.8	1.60	76.5	1.62	201.8	1.62	477.6	1.64	373.3	1.62	285.5
5.0	1.56	55.6	1.60	146.1	1.62	245.8	1.62	526.5	1.62	565.8	1.62	482.2
7.5	1.62	87.9	1.64	157.7	1.68	366.4	1.68	730.5	1.68	612.2	1.64	1086.0
10.0	1.64	148.0	1.68	197.1	1.68	449.9	1.71	735.2	1.71	816.0	1.66	1270.0

TABLE 6

Compressive Strength of Sand-Lime Molasses Blocks (as in Table 5) after Washing with Water

2.5	35.7	52.4	86.7	94.1	65.8	60.2
5.0	59.3	111.3	169.2	226.3	211.0	183.0
7.5	91.3	114.7	237.0	380.0	382.0	590.0
1.0	165.6	140.1	221.2	442.0	485.6	707.0

6.2. It appears from Table 6 that compressive strength falls considerably as a result of washing. A compressive strength of 250 lb per sq. in. can be attained only with 7.5 per cent lime-sand mixture when compacted with at least 5 per cent molasses. For a compressive strength of 400 lb per sq. in., 8 per cent molasses by weight is required.

7. COMPRESSIVE STRENGTH OF BLOCKS WASHED WITH WATER SATURATED WITH CARBON DIOXIDE

7.1. Under field conditions the water may frequently be mixed with CO_2. Experiments were, therefore, undertaken to investigate the loss in compressive strength when washed with water containing CO_2. Water saturated with CO_2 was obtained by carefully adjusting the flow of the two ingredients. The set up for the experiment is shown in Fig. 5

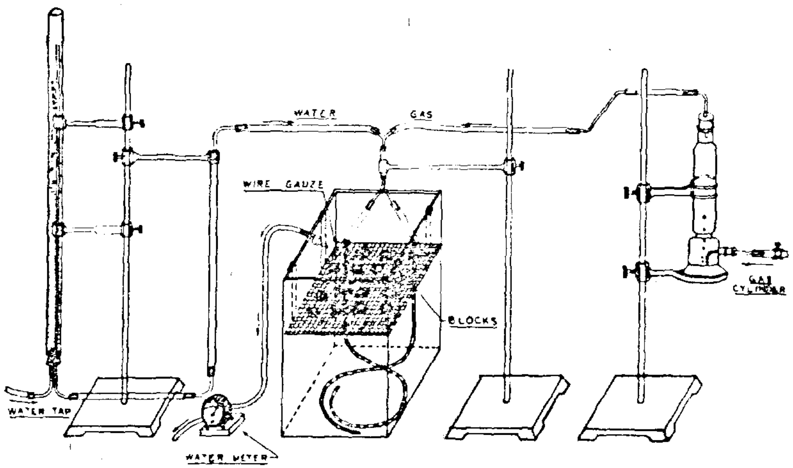

Fig-5

Set up of the Experiment for Washing of Sand-Lime Molasses Blocks with Water Saturated with CO_2.

7.2. Since the test is far more severe than the conditions in the field, it was considered advisable to know the maximum strength that can be attained with such mixtures, under laboratory conditions. Therefore blocks with a 10 per cent lime-sand mixture was compacted with 10 per cent molasses only, and were

tested after the necessary curing and drying. The results are given in Table 7.

TABLE 7

Compressive Strength of 10 per cent Lime-Sand Mixtures compacted with 10 per cent Molasses, and washed with water saturated with CO_2.

No. of days	Compressive strength* lb per sq. in.
0	1055
1	497
3	458
7	416

*Average of 3 tests.

It will appear from the above results that the reduction in the compressive strength is slightly more, when the water used for washing the blocks is saturated with CO_2 than when washed with water alone. The fall in compressive strength is maximum in the first day and subsequent washing brings only a small change.

8. RESISTANCE TO THE SOFTENING ACTION OF WATER

8.1. Blocks made of sand-lime and molasses, of the proportions given in Table 5, were prepared and put under flowing water for 3 days.

8.2. It was noticed that lime and unreacted molasses started coming out gradually on the first two days when the blocks were in contact with water.

8.3. Blocks made with 2.5 lime-sand mixture compacted with 10 per cent molasses, disintegrated. Those with 2.5 per cent lime-sand mixture compacted with other percentages of molasses appeared rather soft. The remaining blocks looked sound.

9. RESISTANCE TO THE SOFTENING ACTION OF WATER SATURATED WITH CO_2

Blocks prepared for the experiment described in para. 8.1 were suspended in moving water saturated with CO_2. It was noticed that at the end of 3 days the blocks prepared with 2.5 and 5 per cent lime-sand mixtures compacted with different percentages of molasses disintegrated. The remaining blocks apparently looked sound.

10. INFERENCE

The study shows that a reasonably good compressive strength and a fair resistance to the softening action of water can be obtained with mixtures of lime-sand and molasses. Therefore, lime-sand mixed with molasses have a potential use for base courses for roads in sandy areas and for the construction of low cost houses.

11. CHEMISTRY OF THE REACTION BETWEEN LIME AND MOLASSES

11.1. The chemical reactions that take place when lime and molasses are mixed together, dried subsequently and washed with simple water, or water saturated with carbon dioxide are discussed in the following paras.

11.2. Molasses is mostly a mixture of glucose ($C_6H_{12}O_6$) and sucrose ($C_{12}H_{22}O_{12}$). Also small quantities of some inorganic salts and organic compounds (non-sugars) are present.

11.3. Action of Lime on Molasses

Theoretically, when lime $Ca\ (OH)_2$ is added to molasses solution, at temperature not exceeding $10°\ C$, mono and di-saccharates are formed. These compounds are insoluble (in water at temperature below $10°\ C$) but are soluble at room temperature. On the other hand when lime is added to a hot solution of molasses, mostly tri-calcium saccharates, ($C_{12}H_{22}O_{11}\ 3\ Cao$) are formed with small quantities of mono and di-saccharate, depending upon the amount of lime used. Glucose is generally destroyed when allowed to react with lime at high temperature. Tri-calcium saccharate is a gelatinous mass which is insoluble at the room temperature but become soluble as the temparature falls below $5°C$.

The reaction between lime and molasses is rather complicated and depends upon (i) proportion of lime to molasses (ii) temperature of reaction, and (iii) the duration of the reaction. It is, therefore, not possible to judge correctly the various reactions which take place, under the conditions in which the present study has been conducted. Attempt has however, been made to find out the nature of reaction by determining the solubility of the saccharates formed, at various temperatures. Blocks about one year old were taken for the tests. The composition of the sand lime molasses mixture which was used for making the blocks is given below:

Sand	180 gm	
Lime	20 gm	
		(15.71 gm $Ca\ (OH_2)$
		(1.76 gm $CaCO_3$.)
Molasses	20 gm	

The solubility of the saccharates formed in the sand lime molasses blocks was determined at various temperatures and is given below:

	Sucrose in gm.	Glucose
1. Solubility in water at $5°\ C$	Nil	Nil
2. Solubility in water at $25°\ C$	3.466	Nil
3. Solubility in water saturated with CO_2 at $25°\ C$	5.4	Nil
4. Solubility in dilute HCl of residue from 3	0.5 (5.9) in all	Nil

11.4 Effect of water saturated with carbon dioxide on the lime content.

Besides reacting with sucrose from the lime-sand molasses mixture, the carbon dioxide also converts $Ca\ (OH)_2$ into $CaCO_3$. The rate at which the reaction is taking place can be judged from the following table:

	$Ca\,(OH)_2$ gm.	$CaCO_3$ gm.	Loss of $Ca(OH)_2$ by washing gm.
Amount of lime added in the mixture	15.71	1.76	
Lime present after mixing with molasses and drying (one year only)	10.79	1.72	
After washing with CO_2 water for one day	6.74	5.84	1.01
After washing with CO_2 water for 3 days	6.52	6.02	0.08
After washing with CO_2 water for 7 days	6.23	6.06	0.26

The washing was not carried out beyond 7 days as it was considered that even 7 days washing is much too severe a test for the conditions prevailing in the field.

11.5. Discussion of the Results

The solubility tests show that there is no glucose present in any of the water extracts. This means that glucose is destroyed after some time when lime and molasses are mixed in the proportion used in the study. The extract at 5° C does not contain any sucrose; which is contrary to the expectations because molasses and lime were mixed at room temperature and it was expected that tri-calcium saccharate is formed which should have been soluble in water at 5° C. It is, therefore, not possible to say correctly the type of compound that is formed. Since water at room temperature could extract a little more than half the sucrose from the saccharates formed, it shows that dicalcium saccharates are formed. In addition to this, there is yet another compound of calcium with sucrose which is broken down only with CO_2.

Reaction with CO_2. It appears from the results that about 5 gm of $Ca\,(OH)_2$ is used for reacting with 20 gm of molasses containing 6.11 gm sucrose and 2.76 gm glucose.

Except for small quantity of $Ca\,(OH)_2$ which is washed away bodily in the first one day washing, the remaining $Ca(OH)_2$ gets converted into carbonates till at the end of seven days the concentration of both $Ca\,(OH)_2$ and $CaCO_3$ is the same.

11.6. Chemical Reaction and the Compressive Strength of the Blocks

The compressive strength of mixtures of sand and lime alone is very poor, but as a result of addition of molasses, the compressive strength increases considerably. This can be due to formation of calcium saccharates and gelatinous matter in the molasses. When the blocks are washed with water, the free sugar, and other soluble matters of the molasses are washed out. The compressive strength of the washed block can therefore be considered mainly due to insoluble saccharates which are not easily washed out from the block and partly due to setting of lime. When the blocks are washed with water saturated with CO_2, the saccharates decompose, but the reaction is not complete even under exhaustive laboratory treatment. As a result of passing CO_2, $Ca(OH)_2$ is converted into $CaCo_3$. The compressive strength of the blocks washed with water saturated with CO_2 can therefore be partly due to undecomposed sacchartes and partly due to $CaCO_3$ formed in the blocks.

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