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Paper No. 225

"STRESSES IN THE CORNER REGION OF THIN BONDED CONCRETE ROAD SLABS"

By

S. KOTESWARA RAO,* B.E., M. TECH.,

&

V. VENKATASUBRAMANIAN,* M.A., B.E., M.S., A.M.I.E.

CONTENTS

		Page
1.	Introduction ...	149
2.	Details of Experiment ...	150

SYNOPSIS

Very little information is available regarding load stresses in concrete road slabs constructed under Indian conditions, where thin slabs on a well consolidated water-bound macadam base are used. Experiments on load stresses in the corner region of a thin bonded concrete road slab constructed under field conditions were taken up at the Indian Institute of Technology, Kharagpur, and the results verified with the existing theories. It was found that observed corner load stress agreed closely with Westergaard's theoretical values using his corner load formula. It was also found that in the vicinity of the corner bisector where the observed strains were maximum, the locus of the maximum strain and the shape of the actual load break were nearly straight lines, thus justifying Westergaard's assumption. The studies revealed that Westergaard's corner load formula can be used in the design of thin bonded concrete road slabs.

1. INTRODUCTION

Thin concrete road slabs of 4 to 6 in. of uniform thickness are generally used for road work in India. These thin slabs of 4 to 6 in. thickness are laid over well consolidated water bound macadam base. There is no rational basis for the design of these slabs, except that they have served their purpose well, the macadam base below the concrete providing a fairly rigid and unyielding base. The comparatively little volume of information available regarding observed stresses in concrete road slabs refers to thicker slabs laid directly on a densified natural subgrade, having

* Indian Institute of Technology, Kharagpur.

a subgrade modulus not exceeding about 250 pounds per cu. in. With a view to study the stresses set up in the corner region of a thin bonded concrete slab, actual experiments were undertaken at the Indian Institute of Technology, Kharagpur, and the results are reported in this Paper.

2. DETAILS OF EXPERIMENT

2.1. To simulate field conditions, a 33 ft $\times$ 12 ft $\times$ 4 in. thick slab was laid in an open space. The subgrade was of fine silty sand, hard as rock in summer and of very soft consistency during the heavy monsoons. After removing the top vegetation, the natural soil was excavated to a depth of 9 inches, and reconsolidated well, with a 8-ton power roller, to compact and level the surface. A plate bearing test on this prepared subgrade gave a subgrade modulus of 1120 lb per cu. in. This high value represented the best summer condition of the subgrade. On this 35 ft $\times$ 14 ft prepared subgrade, a 6 in. thick soling using 6 in. size laterite stones was laid and thoroughly consolidated with the 8-ton power roller. On this soling, 4 in. thick water-bound macadam base was constructed using $1\frac{1}{2}$ in. size hard granite stones. Special care was taken during rolling that the required thickness was maintained at the edges. After nearly three months of air curing, early in 1959, the macadam base was thoroughly cleaned, a rich cement slurry was spread over the cleaned base and 4 in. thick concrete slab was laid. The concrete used gave a 28 days cube strength of 2,800 lb per sq. in. The poor strength of the concrete was due to the fact that the non-tilting mixer available and used necessitated a higher water-cement ratio with a consequent reduction in strength. Nonetheless, it was felt the low strength might not seriously affect the general trend of the conclusions. The general layout of the slab is given in Photo 1.

2.2. Copper-constantan thermocouples installed at different depths of the slab provided data regarding the diurnal variations of temperature at different depths during the summer and winter of 1959. The results of these observations were reported separately elsewhere.[1]

2.3. Studies regarding the load stresses in the corner region were taken up in January 1960. Loads were applied at the corners by means of a hydraulic jack acting against the under-side of a long 4 in. by 9 in. R.S. joist spanning in a direction[1] parallel to the transverse edge of the slab, and bolted securely to two massive reinforced concrete blocks founded away from the longitudinal edges of the slab. The diameter of the circular plate for the load test was arrived at as follows. Messrs Firestone Rubber and Tyre Company recommended a maximum wheel load of 5,150 lb and tyre

inflation pressure of 75 pounds per sq. in. Taking the contact pressure between the wheel load and the road surface as approximately 1.3 times the inflation pressure[2], the contact pressure worked out to 97.5 lb per sq. in. and the diameter of the contact area, assuming a circular shape, came to 8.125 in. Accordingly, a circular mild steel plate 5/8 in. thick, and $8\frac{1}{8}$ in. diameter with machine finished edges, was selected as the bearing plate for load applications. A piece of fibre board about 1/2 in. thick, cut into a circular shape of the same diameter, was kept under the bearing plate at the time of application of loads in order to distribute the loads uniformly over the contact area. The position of the bearing plate for the load applications was such that the two edges of the slab forming the corner were tangential to the plate. This meant that the centre of the loaded area was 5.66 inches from the corner.

2.4. Corner load deflections at various points along the corner bisector were recorded with the help of dial gauges reading upto 0.0001 of an inch. Also the surface strains were measured along 5 radial lines emerging from the loaded corner by means of mechanical Demac strain gauge having a least count of 0.0000985 of an inch. The 5 radial lines were the two edges forming the corner, the corner bisector, and the two intermediate lines $22\frac{1}{2}°$ from the corner bisector and on either side of it. This can be seen in Photo 2. The deflection measurements of the slab were used to calculate the modulus of subgrade reaction from Westergaard's formula[3]

$$Z = \frac{P}{Kl^2}\left(1.1\, e^{-x/l} - \frac{a_1}{l} \times 0.88\, e^{-2x/l}\right) \quad \ldots\ldots\ldots\ldots \quad 1.$$

in which

Z = deflection of the slab in inches.

P = load near the corner in lb.

x = distance from the corner to any point on the corner bisector in inches.

a_1 = distance of the centre of gravity of the applied load from the corner in inches.

l = radius of relative stiffness in inches, and

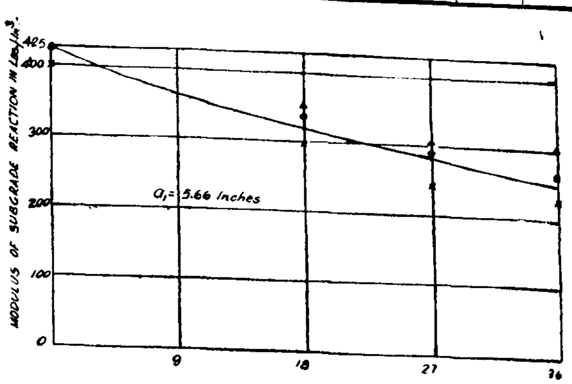
K = subgrade modulus in lb per cu. in.

These results are given in Table 1 and plotted in Fig. 1. It can be observed that the value of the subgrade modulus is not constant for any point under the load as has been assumed by Westergaard. The value of the subgrade modulus varies from a maximum of 425 lb per cu. in. to a minimum of 225 lb per cu. in. It appears that the fundamental assumption for Westergaard's corner load formula is not fulfilled in actual practice.

TABLE 1

Measured corner load deflections (z in inches), and the calculated corresponding subgrade modulii (K in lb per cu. in.) for different corner loads

Load applied in pounds	At the corner		18 in. from the corner		27 in. from the corner		36 in. from the corner	
	Z	K	Z	K	Z	K	Z	K
2,205	0.0213	400	0.0101	295	0.0067	240	0.0044	225
4,410	0.0408	425	0.0180	335	0.0116	290	0.0075	260
5,513	0.0502	425	0.0221	350	0.0140	300	0.0079	300



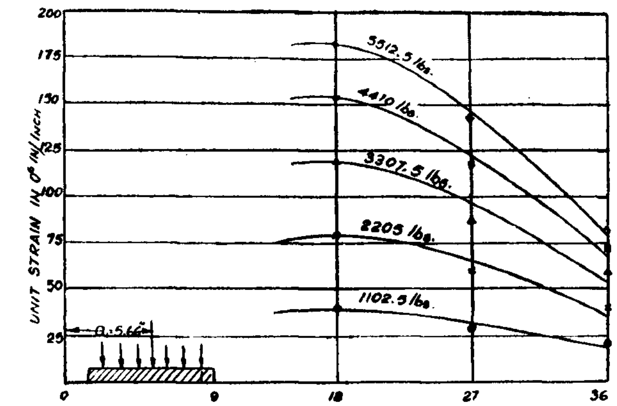
DISTANCE IN INCHES
FROM THE CORNER ALONG THE CORNER BISECTOR

Fig. 1. **Calculated Subgrade Modulus from observed Deflections.**

2.5. A plate-bearing test conducted in October 1959 after the monsoons on the open base course outside the slab portion gave a value of 405 lb per cu. in. for the subgrade modulus. This value is higher than the average value of the modulus obtained from the deflection measurements in the corner region of the slab made in January 1960. It is probable that the moisture content in the base course under the slab at the time of deflection measurements may be more, leading to a lower value of the subgrade modulus; however,

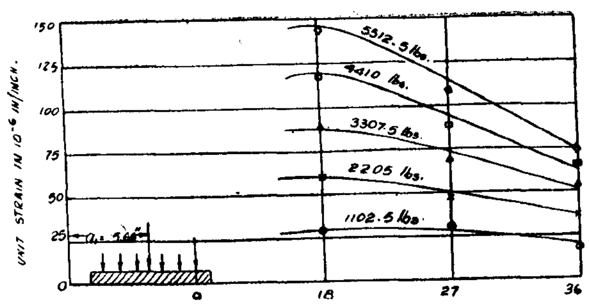
this requires further observation before definite conclusions can be drawn.

2.6. Strains due to the corner loading were measured along each of the 5 radial lines at points 18 inches, 27 inches, and 36 inches from the corner. On the three intermediate radial lines at each of the points, strains were measured in three directions forming a rosette, whereas along the edges the radial strains alone could be measured. The observed radial strains under the loads of 1,102, 2,205, 3,307, 4,410, and 5,512 lb are shown in Figs. 2 to 6 and



DISTANCE FROM CORNER OF SLAB IN INCHES

Fig. 2. **Measured Strains along 45° Radial Line**



DISTANCE FROM CORNER OF SLAB IN INCHES

Fig. 3. **Measured Strains along 22½° Radial Line**

given in Table 2. Recorded readings are the average of three repetitions of each load. These figures show the variation of the radial strain as its distance increases from the corner. It is to be observed that the maximum radial strain occurs on each of the 5 radial lines at distances ranging from 16.5 in. to 20 in. from the corner, It can also be seen that the strains occurring on the 45° radial line are greater than those at the corresponding points on the other radials. The maximum radial strain on the 45° radial occurs at a

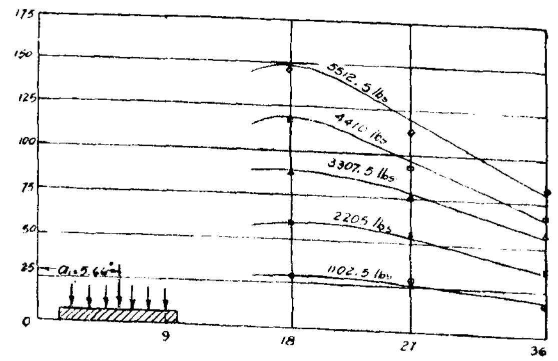


Fig. 4. **Measured Strains along 67½° Radial Line**

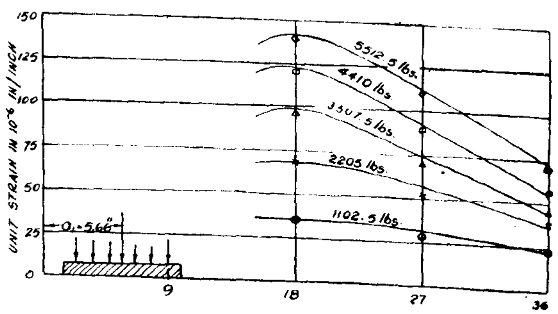


Fig. 5. **Measured Strains along Longitudinal Edge**

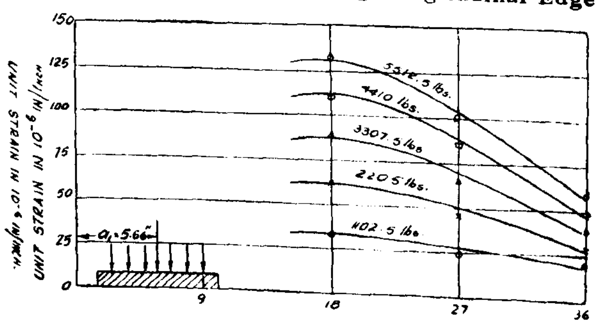


Fig. 6. **Measured Strains along Transverse Edge**

distance of 16.5 to 18.5 inches from the corner for the different loads. This agrees closely with Westergaard's supposition that the maximum stress occurs at a distance approximately equal to:

$$x_1 = 2\sqrt{a_1 . l}$$

where x_1 = the distance of the point of the maximum stress from the corner.

a_1 = the distance of the centre of the loaded area from the corner.

l = radius of relative stiffness.

In this particular case, the value of x_1 worked out to 19.0 in. From the above strain measurements an iso-strain diagram is drawn showing the lines of equal radial strain in the corner region of the slab. This is given in Fig. 7. This diagram helps us to verify Westergaard's assumption that when the corner is loaded, the maxi-

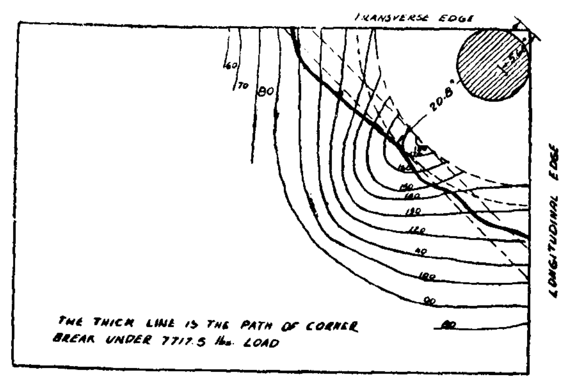


Fig. 7 Iso-strain Diagram Showing Radial Unit Strains in 10^{-6} in/Inch Under a Load of 5512.5 lb.

mum tensile stress is distributed uniformly along a line normal to the corner bisector. An examination of the diagram shows this assumption is not fully justified. A line drawn normal to the corner bisector at the point of maximum strain cuts across the iso-strain lines. It can also be seen from this figure that the strains are greater near the vicinity of the bisector than at the edges; so it is evident that the locus of the maximum strain is not a straight line normal to the corner bisector but rather a curved line which bends towards the

TABLE 2

Measured radial strains over the five radial lines.

Recorded readings are the averages of three cycles of loading

Applied load in pounds	Transverse edge			22½° radial from transverse edge			45° corner bisector			67½° radial from transverse edge			Longitudinal edge		
	18 in. from corner	27 in. from corner	36 in. from corner	18 in. from corner	27 in. from corner	36 in. from corner	18 in. from corner	27 in. from corner	36 in. from corner	18 in. from corner	27 in. from corner	36 in. from corner	18 in. from corner	27 in. from corner	36 in. from corner
1102	32.8	26.0	19.7	29.6	29.6	19.7	39.4	29.6	19.7	29.6	29.6	19.7	36.0	29.6	23.0
2205	62.4	45.9	29.6	59.1	49.3	36.0	78.8	59.1	39.4	59.1	55.7	36.0	69.0	52.5	39.4
3307	88.6	65.6	39.4	88.6	69.0	52.5	118.2	88.6	59.1	88.6	75.5	55.7	98.5	72.2	49.3
4410	111.7	85.4	49.3	118.2	88.6	65.6	154.2	118.2	72.2	118.2	95.2	69.0	121.5	91.9	59.1
5512	131.3	102.2	59.1	144.3	108.3	75.5	183.8	144.3	82.2	147.8	114.8	82.0	141.2	111.7	72.2

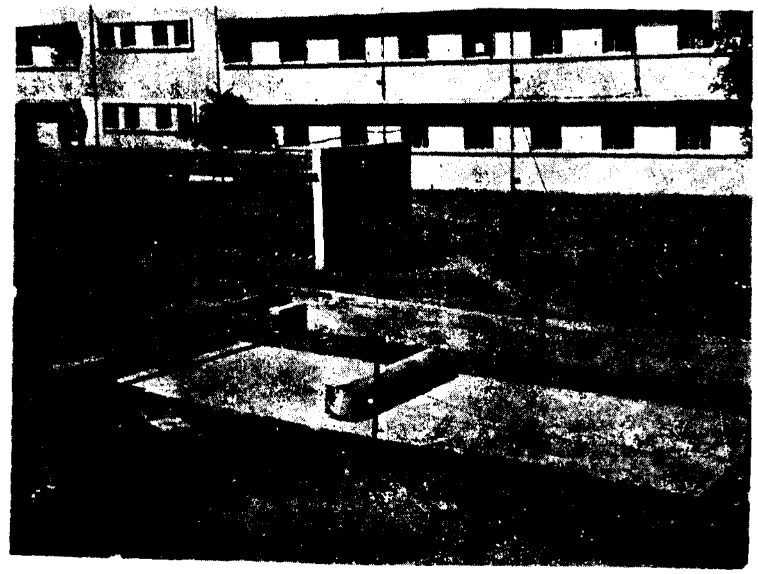


Photo 1. **General layout of slab**

Photo 2. **Layout of strain rosettes in the corner region**

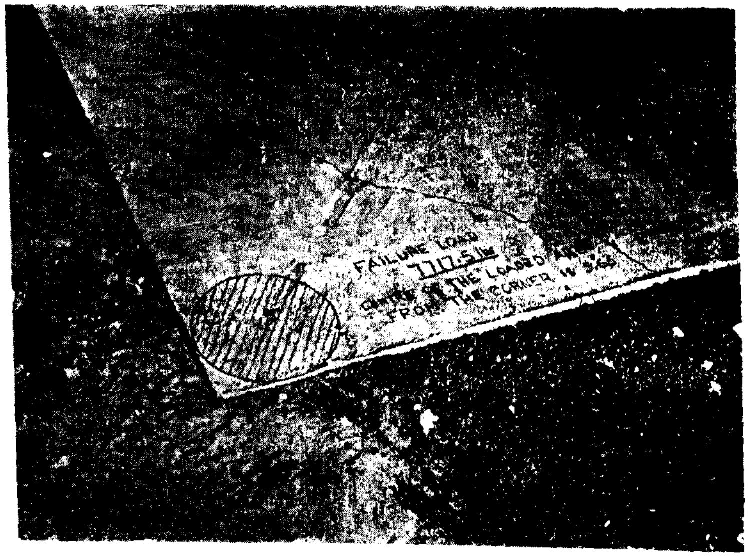


Photo 3. **Shape of the corner break**

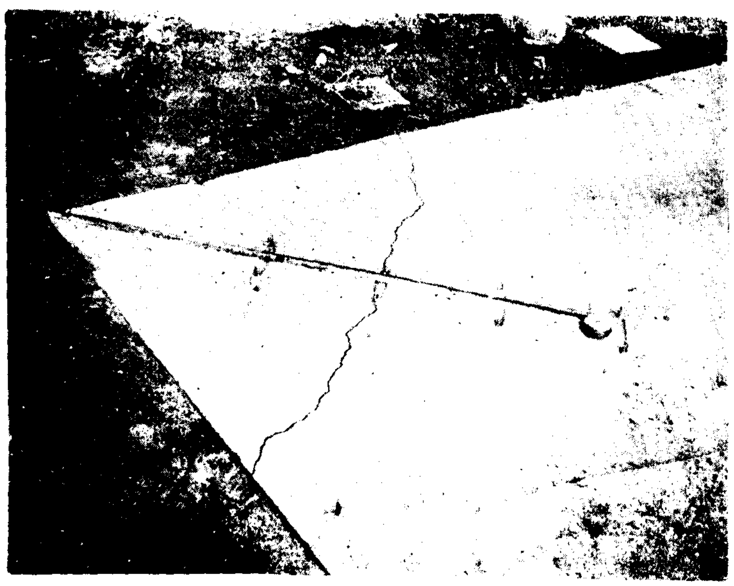


Photo 4. **Shape of the adjacent corner break**

corner as it approaches the edges of the slab, and the strain is not uniformly distributed over this path but is more in the vicinity of the bisector than at the edges. However, actual observations of the shape of the corner breaks indicate that it does not coincide with the locus of maximum strain in the slab. This may be due to the fact that the break is influenced by other factors like the non-uniform strength characteristics of the concrete, subgrade, etc.

2.7. After the strain measurements were completed, the corner was loaded to failure which occurred under a load of 7,717 lb. The path of the corner break is shown in Photo 3 and Fig. 8. In the vicinity of the corner bisector, it is more or less a straight line normal to the bisector but tends to curve towards the corner as it approaches the edges. It appears, therefore, that the locus of the maximum strain under a corner load may lie anywhere between a

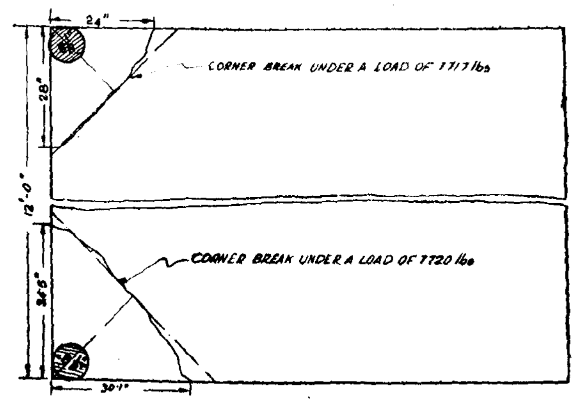


Fig. 8. **Paths of the Corner Breaks under applied Corner Loads**

straight line normal to the corner bisector and a circular curve having the corner as centre and the former line as its tangent. The adjacent transverse corner was also loaded to failure, and it is interesting to note that the break occurred under the same load of 7,720 lb. The shape of this break is given in Photo 4. These observations are in close agreement with that of Spangler[3].

2.8. Using the strain measurements in the strain rosettes at different points on the radial lines, the principal strains and stresses at those points were computed [4]. The computed stresses for the load of 5,512 lb are only shown here as this load is nearer to the suggested

2

load of 5,150 lb. These measured stresses were then compared with the stresses calculated using the formula developed by Spangler [3].

$$\sigma = \frac{CP}{h^2}\left[1 - \frac{a_1}{x_1} - 2.2\left\{1 - e^{-x_1/l}\left(\frac{x_1}{l}+1\right)\right\}\right.$$
$$\left. + 4.4\,\frac{l}{x_1}\left\{1 - e^{-x_1/l}\left(\frac{x_1^2}{2l^2}+\frac{x_1}{l}+1\right)\right\}\right]$$

in which σ = The stress, h = Depth of the slab in inches and C = a co-efficient,

the other notations being as given in the previous equations. This equation gives the maximum probable stress when the value of the coefficient C equals 4.6, and minimum probable stress when C equals 3.0. The measured stresses and the maximum and minimum probable stresses calculated from the above equation are given in Table 3, and plotted in Fig. 9 which shows that the measured stresses lie in between the maximum and minimum probable stresses and are nearer

TABLE 3

Computed and calculated stresses in p.s.i. along the corner bisector.
Recorded readings are the averages of 3 cycles of loading.

Distance of the point from the corner inches	Observed	Spangler's formula	
	Computed maximum stress lb/sq. in.	Calculated maximum stress lb/sq. in.	Calculated minimum stress lb/sq. in.
14	—	663	432
18	491	666	435
20	—	648	423
22	—	595	388
27	379	474	309
32	—	362	236
36	224	258	169

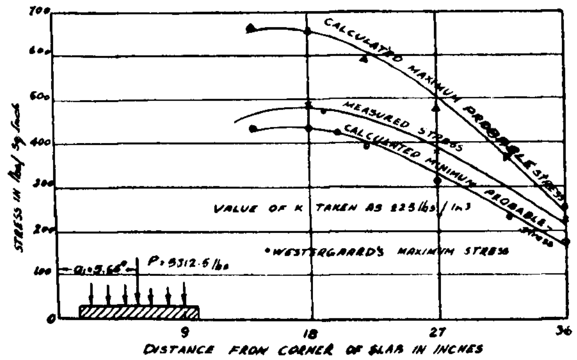


Fig. 9. **Variation of stresses along the corner bisector**

to the minimum probable stresses. The maximum stress according to Westergaard's formula

$$\sigma_c = \frac{3p}{h^2}\left[1 - \left(\frac{a_1}{l}\right)^{0.6}\right]$$

is also shown in the same figure. It is interesting to note that the maximum Westergaard's stress which is equal to the 480 lb per sq. in. compares very favourably with the maximum measured stress of 491 lb per sq. in. This close agreement between the measured maximum stress and Westergaard's maximum stress, and the approximately straight line path of the corner breaks in the region of the corner bisector suggests that Westergaard's equation for the corner stress can be used to give reasonably accurate results in the case of thin bonded concrete pavements.

2.9. It appears that Goldbeck's corner formula cannot be applied in our case as the pavement is bonded to the base course, and thus might not have completely lost support even in the corner region. The high values of the subgrade modulus obtained in the corner region from the deflection measurements and the comparatively low values of the strain measurements under the loads justify this assumption.

2.10. It may, however, be questioned that the reason for the occurrence of such low values of strains and deflections is due to the fact that the test slab was not subjected to a large number of load repetitions which a pavement in the field would experience. But the load tests had been conducted on the test slab after a lapse of one year, and during this period the slab and the subgrade had been subjected to intense dry and wet weather conditions and great fluctuations in seasonal and daily temperatures; these must have caused the slab to warp up and down several times and must have induced stresses fairly comparable to the load stresses; so it is reasonable to assume that the test slab had been subjected to intense stress situation comparable to load stresses; and the low values of strain and deflection measurements may be attributed to the fact that the base had not lost contact with the pavement even in the corner region. The Goldbeck's formula

$$f = \frac{3P}{h^2}$$

where f is the modulus of rupture of the concrete in lb per sq. in.

gives a stress value of 1,035 lb per sq. in. which is more than double the maximum measured stress under the load of 5,512 lb. This clearly indicates that Goldbeck's formula cannot be applied to thin bonded concrete pavements.

REFERENCES

1. V. Venkatasubramanian, "Temperature Stresses in Thin Bonded Concrete Pavements". I.I.T., Kharagpur.—Paper presented at the International Symposium on Concrete and Reinforced Concrete in Hot Countries, July 1960. Haifa, Israel.
2. D.S.I.R., London "Soil Mechanics for Road Engineers".
3. Spangler, M.G. "Stresses in the Corner Region of Concrete Pavements"—Iowa Engineering Experiment Station, Bulletin No. 157, 1942.
4. Frocht—"Photoelasticity" Vol. 1. John Wiley & Sons, New York.

Paper No. 226

"STUDY ON THE STABILITY OF WELL FOUNDATIONS FOR MAJOR BRIDGES"

By

A. BANERJEE*, B.E., A.M.I.E.

&

S. GANGOPADHYAY†, B.SC., B.E., C.E., A.M.I.E.

CONTENTS

		Page
1.	Introduction ...	164
2.	Vertical and Horizontal Forces Acting on a Well ...	165
3.	Assumptions ...	165
4.	Horizontal Soil Reaction ...	166
5.	Deduction of Formulae Allowing no Plastic Flow ...	167
6.	Evaluation of Base Pressure ...	173
7.	Determination of the Values of Co-efficient of Vertical and Horizontal Subgrade Reactions ...	174
8.	Deduction of Formulae Considering Partial Plastic Flow ...	176
9.	Examples ...	179
10.	Factor of Safety ...	182
11.	Conclusion ...	183

SYNOPSIS

Based on assumptions specified in the Paper, formulae have been worked out taking into consideration various combinations of vertical and horizontal forces including active and passive earth pressures. Use of formulae has been explained by working out examples.

* Executive Engineer (Roads), West Bengal.

† Assistant Engineer. (Roads), West Bengal.

1. INTRODUCTION

1.1. Design of well foundations for major bridges in India is a great problem to the bridge engineers of India. The reason is not far to seek. Whereas in most of the progressive countries of the West, the rivers scour very little and the bridges can easily be founded on hard strata or rock available very near the bed, the foundation of majority of bridges in India must necessarily be taken down deeper, because of the very great scour and non-availability of sand strata nearer the bed. As a result, not much work on the problem of design of well foundations has been carried out by authors in the western countries particularly when such foundations are deep and rest on sand strata.

1.2. The problem arises due to the surrounding earth material round a well, offering resistance to the overturning of a well. This requires understanding of the behaviour of this earth material before an evaluation of its resistance can be made. A large field has been opened out by the introduction of soil mechanics in the field of science through the pioneering efforts of Karl Terzaghi and others.

1.3. Till recently the design of well foundations was done in this country with the assumption that soil surrounding a well below the scour depth imparts certain fixity to the base and only an assumed fraction of the full value of horizontal forces is effective in imparting overturning moment at the base.

1.4. Very recently there has been a realisation amongst bridge engineers to approach the problem more mathematically. So far they have been guided in this matter by the design of posts or piles or free rigid bulk heads enunciated by Terzaghi and others. It is, however, apparent that the characteristics of a heavy and rigid well subjected to an appreciable sliding resistance at the base are different from those of piles or posts or bulk heads, which are comparatively light.

1.5. The Authors have, in this article endeavoured to deal briefly with the different cases of the foundation characteristics of a well relating to the surrounding earth material. Certain formulae have been worked out which may help in the solution of these cases. Use of the formulae has been explained by working out an example.

2. VERTICAL AND HORIZONTAL FORCES ACTING ON A WELL

2.1. It is known that in addition to the self weight and buoyancy, a well carries the dead load of the superstructure, bearings, pier and is also liable to the following horizontal forces:

(i) braking and tractive effort of the moving vehicles;

(ii) force on account of resistance of the bearings against movement due to variation of temperature;

(iii) force on account of water current;

(iv) wind force;

(v) seismic force;

(vi) earth pressure;

(vii) centrifugal forces, etc.

2.2. It is necessary to find out the magnitude, direction and point of application of all the above forces under the worst possible combination and then to substitute these forces by two horizontal and a single vertical force as shown in the sketch (Fig. 1);

P = Resultant of all horizontal forces in the direction across the pier.

Q = Resultant of all the horizontal forces in the direction along the pier.

W = Resultant of all the vertical forces.

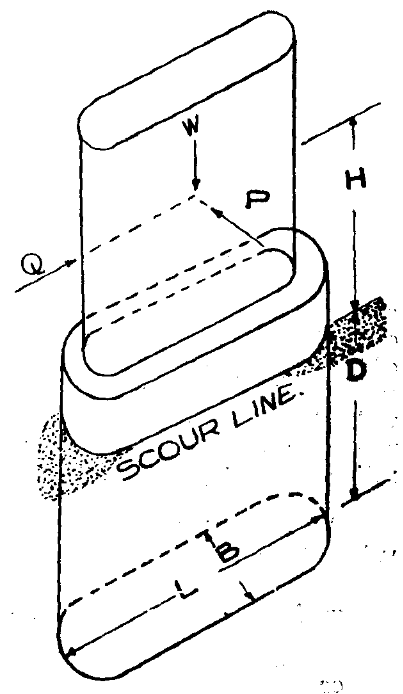


Fig. 1

3. ASSUMPTIONS

The following assumptions will be made in the deductions of formulae that follow:

(1) The well is acted upon by an unidirectional horizontal force, say, a force in the direction across the pier.

(2) The well is founded in sandy stratum.

(3) Resultant unit pressure on soil at any depth is in simple proportion to horizontal displacement.

(4) The ratio between contact pressure and corresponding displacement is independent of the pressure.

(5) The co-efficient of vertical subgrade reaction has the same value for every point of the surface acted upon by contact pressure.

(6) The depth of maximum scour is known.

4. HORIZONTAL SOIL REACTION

Let us consider the relation between pressure and displacement of a vertical wall *ab* buried in sand and acted by a horizontal pull *P* as shown in Fig. 2. At the instant of failure the right hand face of the wall is acted on by the active earth pressure and the left hand face is acted on by the passive earth pressure induced in soil. Assuming hydrostatic distribution of pressure, the resultant unit pressure at a depth z below the surface $p_1 = z\gamma\,(K_P - K_A)$.

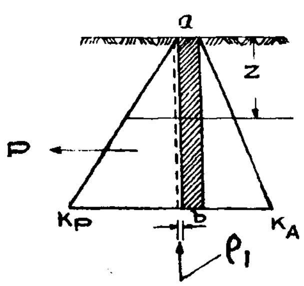


Fig. 2

Where γ is the unit weight of sand and K_A and K_P are co-efficients of active and passive earth pressure. Let p be the load per unit area of vertical surface of sand and ρ be the corresponding displacement of the surface. The values of K_P and K_A, which are again dependent on ϕ the angle of internal friction of sand and δ, the angle of wall friction, may be obtained either from Coulomb's formula with necessary corrections for the surface of sliding or from any other rational formula.

Assuming that ρ_1 is the displacement required to increase the value of resultant unit pressure from zero to p_1,

we have, $p = p_1 \dfrac{\rho}{\rho_1} = \dfrac{\rho}{\rho_1}\gamma z\,(K_P - K_A)$.

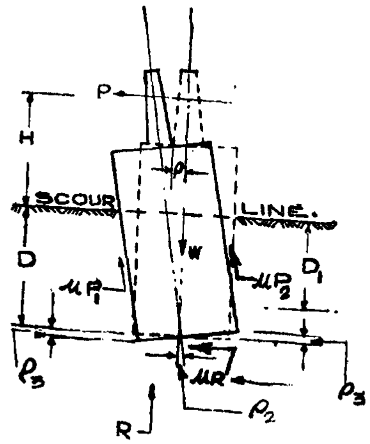
or, $\dfrac{p}{\rho} = mz$,

where $m = \dfrac{\gamma(K_P - K_A)}{\rho_1}$

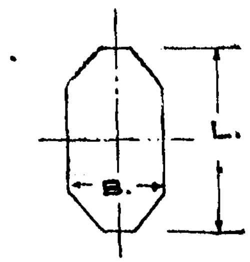
m is an empirical constant which is independent of depth and the ratio between the horizontal unit pressure p and the corresponding displacement ρ represents the co-efficient of horizontal soil reaction. This co-efficient depends not only on the nature of the soil but also on the size and shape of the area which carries the load.

5. DEDUCTION OF FORMULAE ALLOWING NO PLASTIC FLOW

5.1. General case



ELEVATION



PLAN

Fig. 3.

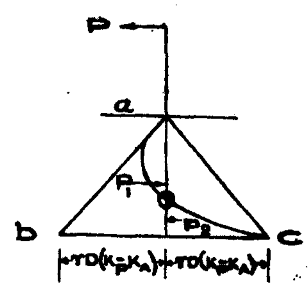


Fig. 4.

Let us consider a rigid well of length '*L*' and width '*B*' acted upon by a horizontal force of *P* per unit length of well and a vertical force W as shown in Fig. 3. Let the point of application of 'P' be at a height 'H' above the designed scour level. Let D be the depth of well below scour line.

Due to the vertical force 'W', there will be a reaction at the base and skin friction at the sides of the well.

Due to the horizontal force 'P' tending to tilt the well towards the left, a resultant passive reaction P_1 (see Fig. 4) will be exerted on the left face. Due to rotation produced by P and P_1 there will be another resultant passive reaction P_2 exerted on the lower part of the right face of the well, provided the well can slide about the base by overcoming the frictional resistance μR of the soil at the base. Thus the well will rotate about a point at a certain depth D_1 from the scour line. This rotation of the well will also be resisted by the skin friction at the side of the well, which without much error may be assumed as μP_1 and μP_2 as shown in Fig. 3. This rotation will also be resisted by a moment acting at the base on account of the downward deflection of the toe and upward deflection of the heel.

Let ρ_u be the uniform vertical displacement of the well due to the resultant vertical force.

Let ρ_1 and ρ_2 be the horizontal displacement of the well at scour level and base respectively due to the horizontal force 'P' and let ρ_3 and ρ'_3 be the vertical displacement of the well at the toe and heel respectively due to this horizontal force.

The difference between ρ_3 and ρ_3' will be negligible and we may assume $\rho_3 = \rho'_3$

Therefore,

$$\frac{\rho_1}{D_1} = \frac{\rho_2}{D\text{-}D_1} = \frac{\rho_3}{\frac{B}{2}}.$$

Evaluation of P_1

Let ρ be the deflection at any depth 'z' below the scour line of an elementary area of depth 'dz' and let the corresponding pressure be 'p', then $p = m\rho z$

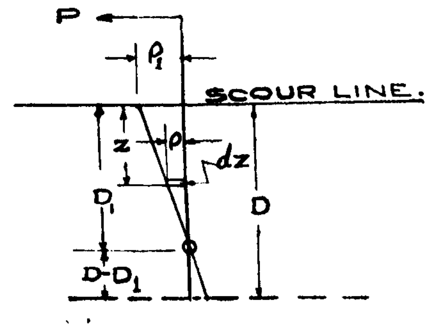


Fig. 5.

$$\text{and } P_1 = \int_0^{D_1} \text{P. dz.}$$

$$= \int_0^{D_1} m\,\rho\,\text{z.dz.}$$

$$\text{Again, } \frac{\rho}{D_1 - z} = \frac{\rho_1}{D_1}$$

$$\therefore \rho = \frac{D_1 - z}{D_1}\rho_1$$

$$\therefore p = m\frac{\rho_1}{D_1}\cdot(D_1 - z)z \qquad \text{.........(1)}$$

$$\therefore P_1 = \int_0^{D_1} m\cdot\frac{\rho^1}{D_1}(D_1 - z)\,zdz$$

$$= \frac{m\rho_1 D_1^2}{6} \qquad \text{.........(2)}$$

Evaluation of P_2

Similarly, when 'z' is greater than D_1, we have,

$$\frac{\rho}{z - D_1} = \frac{\rho_2}{D - D_1} = \frac{\rho_1}{D_1}$$

$$\therefore \rho = \frac{z - D_1}{D_1}\rho_1$$

$$\therefore P_2 = \int_{D_1}^{D} m\frac{\rho_1}{D_1}(z - D_1)\,zdz$$

$$= \frac{m\rho_1}{6D_1}(2D^3 - 3D_1D^2 + D_1^3) \qquad \text{......(3)}$$

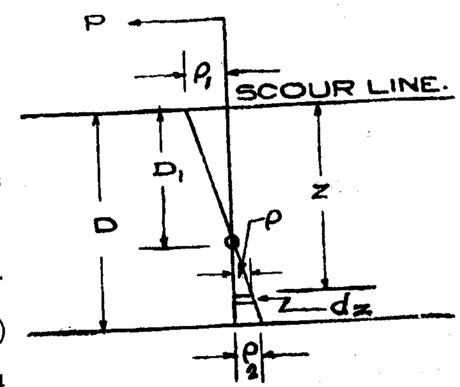


Fig. 6.

Evaluation of moment M_1 produced by P_1

$$M_1 = \int_0^{D_1} pz\,dz.$$

$$= \int_{O}^{D_1} \frac{mP_1}{D_1}(D_1 - z)\, z^2\, dz.$$

$$= \frac{mD_1^{\,3}}{12} P_1 \qquad \ldots\ldots(4)$$

Evaluation of moment produced by P_2

$$M_2 = \int_{D^1}^{D} \frac{m\rho_1}{D_1}(z - D_1)\, z^2 dz.$$

$$= \frac{m\rho_1}{12D_1}(3D^4 - 4D_1 D^3 + D_1^{\,4}) \qquad \ldots\ldots(5)$$

Evaluation of vertical reaction R

Modulus of vertical subgrade reaction,

$$K = \frac{p}{\rho}$$

Where P is the vertical deflection of soil corresponding to vertical load p per unit area,

$$\therefore\ R = \int_{O}^{B_2} p dx \quad \text{(see fig. 7)}$$

$$= \int_{O}^{B_2} K\, \rho_4\, dx.$$

$$= KB.\ \rho_4 \qquad \ldots\ldots(6)$$

Evaluation of moment M_3 produced at the base due to the horizontal force 'P'

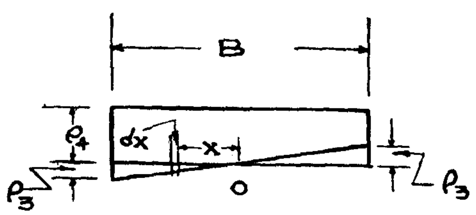


Fig. 7.

Let ρ be the deflection at a distance 'x' from O, the centre of the base as shown in the Fig. 7.

$$\therefore\ \rho = \frac{2\,\rho_3}{B}\, x.$$

$$\therefore\ M_3 = 2\int_{0}^{\frac{B}{2}} pxdx$$

$$= 2\int_{0}^{\frac{B}{2}} K\rho \times dx\ (\because\ p = K\rho)$$

$$= 2\int_{0}^{\frac{B}{2}} K.\ \frac{2\,\rho_3}{B}\, x^2\, dx = K\,\rho_3\, \frac{B^2}{6} \qquad \ldots\ldots\ldots\ (7a)$$

$$\text{Again,}\ \rho_3 = \frac{B\ .\ \rho_1}{2\,D_1}$$

$$\therefore\ M_3 = \frac{K\,B^3}{12D_1}\ .\ \rho_1 \qquad \ldots\ldots\ldots\ (7)$$

The maximum soil pressure at any depth z below the scour level is given by

$$p_z\ \text{max.} = \gamma z\,(K_p - K_A) \qquad \ldots\ldots\ldots\ (8)$$

If no plastic flow is allowed in soil, the horizontal soil reaction p at any depth must not exceed p_z max. for that depth. Hence, it is evident that sand starts to flow as soon as the slope of the pressure parabola at the scour level becomes equal to the slope of the line $a\,b$ whose abscissa represent values of p_z max. (see Fig. 8)

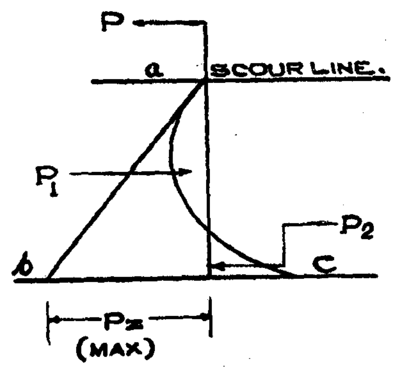


Fig. 8.

Thus from equation (2)

$$\text{we have}\ m\rho_1 = \frac{6p_1}{D^2_{\ 1}}$$

From equation (1)

$$p = \frac{6\rho_1}{D^3_{\ 1}}(D_1 - z)\, z \qquad \ldots\ (9)$$

and thus from equations (8) and (9)

$$\left\{\frac{dp}{dz}\right\}_{z=0} = \frac{6\,P_1}{D_1^{\,3}} \times D_1 = \gamma\,(K_P - K_A)$$

$$\therefore\ P_1 = \frac{\gamma\,(K_P - K_A)\,D_1^{\,2}}{6} \qquad \ldots\ldots\ldots\ (10)$$

$$\text{and}\ \frac{6\,P_1}{D^2_{\ 1}} = r\,(K_P - K_A) = m\rho_1$$

When P_1 is maximum,

From equation (3), $P_2 = \gamma\,(K_P - K_A)\,\dfrac{2D^3 - 3\,D_1\,D^2 + D_1^{\,3}}{6\,D_1}$ .... (11)

From equation (4), $M_1 = \gamma\,(K_P - K_A)\,\dfrac{D_1^{\,3}}{12}$ .... (12)

From equation (5), $M_2 = \gamma\,(K_P - K_A)\,\dfrac{3\,D^4 - 4\,D_1\,D^3 + D_1^{\,4}}{12\,D_1}$ .... (13)

From equation (7), $M_3 = \dfrac{B^3}{12\,D_1} \times \gamma\,(K_P - K_A)\left(\dfrac{K}{m}\right)$ ... (14)

Now the conditions of equilibrium of the well under all the vertical and horizontal forces will be as follows: (see Figures 3 and 4).

$$P = P_1 - P_2 = \mu R \quad \text{......... (15)}$$

$$PH = M_3 + M_2 - M_1 + \mu RD + \mu\,(P_1 - P_2)\,\frac{B}{2} \quad (16)$$

$$\text{and } W = \mu\,(P_1 + P_2) + R \quad \text{......... (17)}$$

Above three equations are valid when deflection of the heel is downwards and also when sliding takes place at the base.

Now by inspection of equation (15), it may be seen that sliding at the base occurs only when $\mu R < P_1 - P_2$.

5.2. **Heavy Wells:** Since for heavy wells 'R', the reaction at base, will be very high, sliding at the base will not take place and if under such circumstances, F denotes the force required at the base to prevent sliding, the well will tilt towards the direction of the applied force as shown in Fig. 9.

Hence $D_1 = D$

$P_2 = 0, \quad M_2 = 0,$

and for maximum value of P, the general equations for equilibrium will be as follows:

$$P = P_1 - F \quad \text{... (18)}$$

$$PH = M_3 - M_1 + FD + \mu\,P_1\,\frac{B}{2} \quad \text{... (19)}$$

$$W = \mu\,P_1 + R \quad \text{... (20)}$$

where $P_1 = \gamma\,(K_P - K_A)\,\dfrac{D^2}{6}$

$M_1 = \gamma\,(K_P - K_A)\,\dfrac{D^3}{12}$ and

$M_3 = \dfrac{B^3}{120}\,\gamma\,(K_P - K_A)\left(\dfrac{K}{m}\right)$

For a given well of known length, breadth and depth, we can solve maximum P, provided the values of μ, ϕ, δ, γ, K and m are known for any particular site.

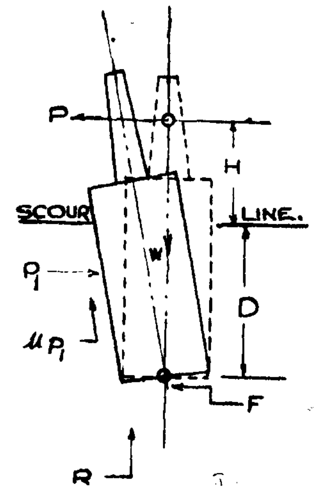


Fig. 9.

6. EVALUATION OF BASE PRESSURE

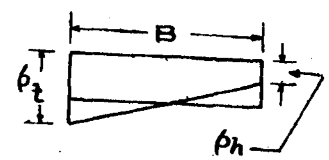


Fig. 10.

Let P_t and P_h be the unit pressures exerted on the soil under the base at the toe and heel respectively.

Then $R = \dfrac{P_t + P_h}{2}\,.\,B$ ... (21)

and $M_3 = \dfrac{P_t - P_h}{3}\,.\,B^2$ ... (22)

Thus P_t and P_h may be evaluated from equations (21) and (22). It may be verified that P_t does not exceed allowable bearing capacity of soil and that P_h is positive, i.e., no tension is developed at the base.

If, however, the well base be not rectangular then,

Let L = The maximum length of the well base.

B = Maximum width of the well base.

A = Area of the well base.

Z = Section modulus of the well base about the axis of tilting.

B_o = Average width of the base.

M_B = Total moment induced in the base due to tilting.

F_1 = Total frictional resistance at the base.

R_1 = Total upward reaction.

W_T = Total vertical force on the well.

Then the general equations for maximum value of P for the conditions of equilibrium may be written as below.

$$PL = P_1 L - F_1 \qquad \ldots\ldots\ldots (23)$$

$$PLH = M_B - M_1 L + F_1 . D + \mu P_1 . L . \frac{B_o}{2} \qquad \ldots\ldots (24)$$

$$W_T = \mu P_1 L + R_1 \qquad \ldots\ldots\ldots (25)$$

If M_3 be the moment exerted on the base,

Let P_t and P_h be the maximum toe and heel pressures,

Then we have,

$$P_t = \frac{R_1}{A} + \frac{M_B}{Z} \qquad \ldots\ldots\ldots (26)$$

$$P_h = \frac{R}{A} - \frac{M_B}{Z} \qquad \ldots\ldots\ldots (27)$$

$$\text{and } M_3 = \frac{B^3}{12_D} . \gamma (K_P - K_A) . \frac{K}{m} = \frac{P_t - P_h}{3} . B^2 \qquad \ldots\ldots (28)$$

whence the values of horizontal force P, maximum toe pressure P_t and maximum heel pressure P_h may be found out.

7. DETERMINATION OF THE VALUES OF CO-EFFICIENT OF VERTICAL AND HORIZONTAL SUBGRADE REACTIONS

7.1. The values of the co-efficient of horizontal and vertical subgrade reactions can either be derived from the results of field tests or estimated on the basis of published observational data given by Terzaghi. Reliable observational data of any other investigator may also be used.

7.2. Co-efficient of vertical subgrade reaction

Experimental investigations have shown that if K_1 is the co-efficient of vertical subgrade reaction of sand for a beam with a width of one ft then k, the co-efficient of vertical subgrade reaction of sand for a width of L ft, may be found by the following formula:

$$K = K_1 \left(\frac{L+1}{2L}\right)^2$$

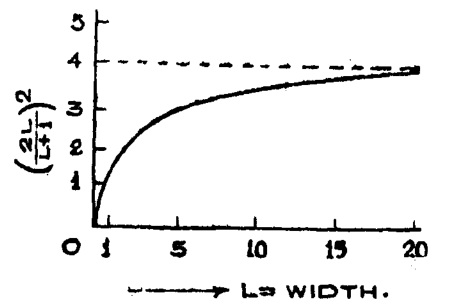


Fig. 11.

The value of K_1 for sand found by Terzaghi[3] is given below:

TABLE 1

Values of K_1 Kip's/cu. ft.

Relative density of sand	Loose	Medium	Dense
1. Dry or moist sand, limiting value.	40 to 120	120 to 600	600 to 2,000
2. Dry or moist sand, proposed value.	80	260	1,000
3. Submerged sand, proposed value.	50	160	600

Knowing K_1 and 'L', i.e., the length of well base, K may be found out.

7.3. Co-efficient of horizontal subgrade reaction

In determining passive resistance of soil, it was assumed that

$$\frac{p}{\rho} = m.\ z. = \text{co-efficient of horizontal subgrade reaction,}$$

where m is a constant. If m_1 is the constant of horizontal subgrade reaction in sand for a beam of one ft width, then the value of m, in sand for a beam of width 'L' ft at right angles to the direction of moment, will be given by

$$m = \frac{m_1}{L} = \frac{A\gamma}{1.35\ L}$$

Value of A given by Terzaghi for a beam of one ft width embedded in sand is as follows:

TABLE 2

Range of values of A

Relative density of sand	Loose	Medium	Dense
Range of value of A	100 — 300	300 — 1,000	1,000—2,000
Adopted value of A	200	600	1,500

TABLE 3

Values of m_1 in kips/cu. ft assuming the unit weight of moist sand to be equal to 100 lb per cu. ft and that for submerged sand equal to 57.6 lb per cu. ft.

Relative density of sand	Loose	Medium	Dense
For dry or moist sand	14	42	112
For submerged sand	8.5	25.6	68.2

8. DEDUCTION OF FORMULAE CONSIDERING PARTIAL PLASTIC FLOW

If the well is acted upon by a horizontal force, which causes plastic flow of soil near the scour line and upto a certain depth, then

the pressure diagram (if no sliding takes place at the base) will be as shown in the Figure 12.

Let ρ_1 be the displacement at the scour level due to a horizontal force P which causes plastic flow upto a depth of d_1 from the scour line.

Let z be any depth between cd and base, and ρ the deflection at a depth z.

Then $\rho = \frac{D-z}{D} \cdot \rho_1$

and $P = m\rho z = m\rho_1 . \frac{D-z}{D} . z$ .... (29)

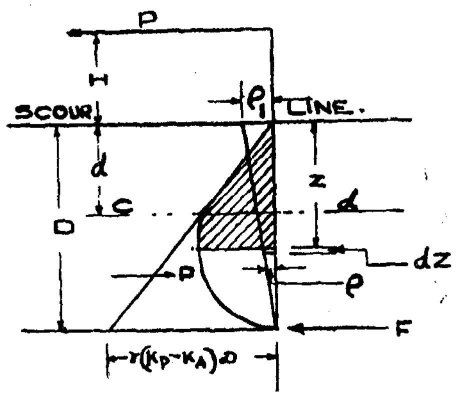


Fig. 12

For a horizontal force P, resultant horizontal earth pressure developed on the left face, (denoted by the hatched area)

$$P_1 = \gamma\,(K_p - K_A) . \frac{d_1^{\,2}}{2} + \int_{d_1}^{D} m\rho_1 . \frac{D-z}{D} . z.\ dz$$

$$= \gamma\,(K_p - K_A) . \frac{d_1^{\,2}}{2} + \frac{m\rho_1}{6D}\,(D^3 - 3D . d_1^{\,2} + 2d_1^{\,3})$$

Hence, $$m\rho_1 = \frac{6D\,\{P_1 - \gamma\,(K_P - K_A) . \frac{d^2{}_1}{2}\}}{D^3 - 3D.\ d_1^{\,2} + 2d_1^{\,3}}$$

Therefore, from equation (29),

$$p = \frac{6D\,\{P_1 - \gamma\,(K_P - K_A) . \frac{d^2{}_1}{2}\}}{D^3 - 3D . d_1^{\,2} + 2d_1^{\,3}} \cdot \frac{(D-z)z}{D}$$

when $z = d_1$, $$P_{z=d_1} = \frac{6D\,\{P_1 - \gamma\,(K_p - K_A) . \frac{d_1^{\,2}}{2}\}}{D^3 - 3D\,d_1^{\,2} + 2d_1^{\,3}} \cdot \frac{(D-d_1).d_1}{D}$$

As plastic flow starts at $z = d_1$, we have

$$P_{z=d_1} = \gamma\,(K_p - K_A) . d_1$$

Therefore, $$P_1 = \gamma\,(K_p - K_A) . \left[\frac{D^3 - 3D . d_1^{\,2} + 2d_1^{\,3}}{6\,(D - d_1)} + \frac{d_1^{\,2}}{2}\right]$$

$$= \gamma\,(K_p - K_A)\,.\,\frac{D^3 - d_1^3}{6(D - d_1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

$$\text{and } m\rho_1 = \gamma\,(K_p - K_A)\,.\,\frac{D}{D - d_1} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Evaluation of the resisting moment about scour line due to ρ_1

$$M_1 = \gamma\,(K_p - K_A)\,.\,\frac{d_1^2}{2}\,.\,\frac{2}{3}d_1 + \int_{d_1}^{D} m\rho_1\,.\,\frac{D - z}{D}\,.\,z^2\,dz$$

$$= \gamma\,(K_p - K_A)\,.\,\frac{d_1^3}{3} + m\rho_1\,.\,\frac{D^4 - 4D\,d_1^3 + 3d_1^4}{12D}$$

Putting the value of $m\rho_1$

we have, $$M_1 = \gamma\,(K_p - K_A)\,.\,\frac{D^4 - d_1^4}{12(D-d_1)} \quad \ldots\ldots\ldots (32)$$

Evaluation of moment exerted on the base

If ρ_3 be the deflection at maximum fibre distance under the base, then

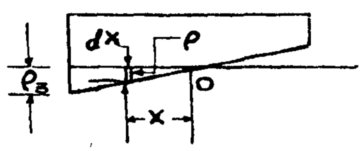


Fig. 13

$$\frac{\rho_3}{\frac{B}{2}} = \frac{\rho_1}{D}$$

Therefore, $\rho_3 = \frac{B}{2D}\,.\,\rho_1$

$$\text{Hence, } M_3 = 2\int_0^{\frac{B}{2}} K\,.\,\rho x\,dx$$

$$= 2\int_0^{\frac{B}{2}} K\,.\,\frac{2\rho_3}{B}\,.\,x^2\,dx$$

$$= K\,.\,\rho_3\,.\,\frac{B^2}{6} = \frac{K\,.\,B^3}{12D}\,\rho_1$$

Putting the value of ρ_1 from equation (31)

$$M_3 = \gamma\,(K_p - K_A)\left(\frac{K}{m}\right)\,.\,\frac{B^3}{12\,(D-d_1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Hence the general equation for equilibrium will be

$$P = P_1 - F$$

$$PH = M_3 - M_1 + FD + \mu P_1 \frac{B}{2}$$

$$W = \mu P_1 + R$$

$$\text{where, } P_1 = \gamma\,(K_p - K_A)\,.\,\frac{D^3 - d_1^3}{6\,(D - d_1)}$$

$$M_1 = \gamma\,(K_p - K_A)\,.\,\frac{D^4 - d_1^4}{12\,(D - d_1)}$$

$$\text{and } M_3 = \gamma\,(K_p - K_A)\left(\frac{K}{m}\right)\,.\,\frac{B^3}{12(D - d_1)}$$

9. EXAMPLES

Problems : Given

W = Effective downward force on the well including self weight
= 1500 tons.

H = Height above the scour level and the point of application of the resultant horizontal force
= 30 ft.

D = Depth of the well below scour level
= 30 ft.

B = Maximum breadth of the well base
= 16 ft.

L = Maximum length of the well base
= 32 ft.

A = Area of the well base
= 457 sq. ft.

Z = 1085 ft^3 (section mod. of base)

r = Unit weight of sand immersed in water
= 57.6 lb per cu.ft.

ϕ = Angle of internal friction of sand
= 30°.

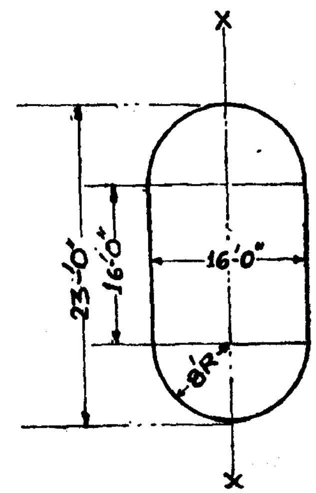


Fig. 14.

To find P and base pressures, if safe bearing capacity of soil at base be 4 tons per sq. ft.

Solution: From Table 1, K_1 = Co-eff. of vertical subgrade reaction for 1 ft width of beam.

= 600 kips per cu.ft.

From Table 2, m_1 = Constant of horizontal subgrade reaction for 1 ft width.

= 68.2 kips per cu. ft.

Let $\mu = 0.5$

Hence, $\gamma (K_P - K_A) = 0.068$ tons per cu. ft.

$$K = 600 \times \left(\frac{32+1}{2 \times 32}\right)^2 = 159 \text{ kips cu. ft.}$$

$$m = \frac{68.2}{32} = 2.13 \text{ kips per ft}$$

$$\frac{K}{m} = \frac{159}{2.13} = 74.65 \text{ ft}$$

(a) $$P_1 = \gamma(K_p - K_A)\frac{D^2}{6}$$

$$= 0.068 \times \frac{30^2}{6} = 10.20 \text{ tons per ft}$$

$$M_1 = \gamma(K_p - K_A)\frac{D^3}{12} = 153 \text{ ft tons per ft}$$

$$M_3 = \frac{B^3}{12D}\gamma(K_P - K_A)\left(\frac{K}{m}\right)$$

$$= \frac{16^3}{12 \times 30} \times 0.068 \times 74.65 = 57.80 \text{ tons per ft.}$$

Referring to general equations (23), (24) & (25) $\therefore$ equation for equilibrium, when soil at the scour level just starts flowing, may be written as, $32P = 32 \times 10.20 - F_1$ ....(1)

$$32 \times 30P = M_B - 153 \times 32 + 30\, F_1 + 0.5 \times 10.20 \times 32 \times \frac{15}{2}$$ ....(2)

where average breadth = 15 ft.

$$1500 = 0.5 \times 10.20 \times 32 + R_1$$ ....(3)

Where F_1, R_1 and M_B are total force at base, total reaction under base and total induced moment in soil below base respectively :

Also, $$p_t = \frac{R_1}{457} + \frac{M_B}{1085}$$ ....(4)

$$p_h = \frac{R_1}{457} - \frac{M_B}{1085}$$ ....(5)

and $$M_3 = 57.80 = \frac{p_t - p_h}{3} \times 16^2$$ ....(6)

Solving equations (3), (4), (5) & (6)

we have P_t = 3.26 ton per sq. ft.

P_h = 2.59 ton per sq. ft.

R_1 = 1337 tons.

M_B = 367 ft tons.

And putting the value of M_B in equation (2) and solving equations (1) and (2), we have,

P = 3.37 tons/rft.

F_1 = 218 tons.

$\mu R_1 = 0.5 \times 1337 = 668$ tons. $> F_1$

Hence total horizontal force that this well can take is = 3.37 × 32 or 108 tons and the maximum base pressure is 3.26 tons per sq. ft which is less than safe bearing capacity of soil.

(b) Considering plastic flow of soil upto one-third of the depth, i.e., d_1 = 10 ft

$$P_1 = \gamma(K_P - K_A)\frac{D^3 - d_1^3}{6(D - d_1)}$$

$$= 0.068 \times \frac{30^3 - 10^3}{6(30 - 10)}$$

= 14.73 tons.

$$M_1 = \gamma(K_P - K_A)\frac{D^4 - d_1^4}{12(D - d_1)}$$

$$= 0.068 \times \frac{30^4 - 10^4}{12 \times 20} = 227 \text{ ft tons.}$$

$$M_3 = \gamma(K_P - K_A)\left(\frac{K}{m}\right) \times \frac{B^3}{12(D - d_1)}$$

$$= 0.068 \times 74.65 \times \frac{16^3}{12 \times 20} = 87 \text{ ft tons.}$$

Equation for equilibrium may be written as,

$$32P = 32 \times 14.73 - F_1 \qquad \text{.........(1)}$$

$$32 \times 30P = M_B - 227 \times 32 + 30\ F_1 + 0.5 \times 14.73 \times 32 \times \frac{15}{2} \qquad \text{.........(2)}$$

$$1500 = 0.5 \times 14.73 + 32 + R_1 \qquad \text{.........(3)}$$

$$p_t = \frac{R_1}{457} + \frac{M_B}{1085} \qquad \text{.........(4)}$$

$$p_h = \frac{R_1}{457} - \frac{M_B}{1084} \qquad \text{.........(5)}$$

$$M_3 = 87 = \frac{Pt - Ph}{3} + 16^2 \qquad \text{.........(6)}$$

Solving the equations, we have

R_1 = 1264 tons.

M_B = 553 tons.

F_1 = 318 tons.

p_t = 3.28 tons per sq. ft.

p_h = 2.26 tons per sq. ft.

P = 4.78 tons per rft.

$\mu R_1 = 0{\cdot}5 \times 1264 = 632$ ton $> F_1$ $\therefore$ no sliding will occur.

Hence, total horizontal force that this well is capable to withstand is = 4.78 × 32 or 153 tons and the maximum base pressure is 3.28 tons per sq. ft which is less than the safe bearing capacity of soil.

10. FACTOR OF SAFETY

10.1. From the method of solution suggested in the preceding pages for the design of well foundations, it will be seen that there are many factors, particularly with regard to the properties of subsoil, which play a dominant part in the calculations.

Since it may not be possible to know the accurate values of γ, ϕ, μ, δ, k, m, etc., due to the various factors involved, it may reasonably be asked what amount of factor of safety should be considered in the calculations.

10.2. It will be seen that in the formulae for general equations, both k and m occur together as k/m. Hence even if correct values

of k and m are not available, the probability of error in evaluation of those two factors is the same and hence no factor of safety is necessary.

10.3 The calculation of horizontal pressure of soil may be greatly affected if the value of ϕ, the angle of internal friction, is not correctly fixed. Usually for sand, the laboratory experiments can give values of ϕ with reasonable accuracy and even if the actual passive resistance is less than that calculated, there is no cause for alarm. First, there is the question of plastic flow before failure may occur and secondly, there is the question of factor of safety allowed in fixing the allowable bearing power of soil at the foundation depth. For safety it is, however, desirable that in actual design the deflection at the scour level and consequently at the top of pier are kept within allowable limits and at the same time plastic flow is restricted within half the grip length.

10.4. The value of δ, the angle of wall friction may, be experimentally determined or may be assumed. It seems rational to adopt a value of δ equal to half of ϕ. For absolute safety, however, δ may be assumed to be equal to zero.

10.5. The value of μ for sand in contact with the skin of the well may vary from 0.45 to 0.56 and may reasonably be assumed as 0.45. In that case, the possible variation will be small and the design will be on the safe side.

10.6. The value of γ, the unit weight, will depend on the state of sand strata at the foundation level and if the same is previously found out by laboratory test of undisturbed soil samples obtained from the proposed foundation level, the error may be negligible.

11. CONCLUSION

11.1. It will be seen that the deduction of the formulae has been based on certain assumptions. Obviously the equations will not hold good if the assumptions are not true.

11.2 If a well is acted upon by forces in the direction both along and across the flow of water, the equations may still be applied by resolving the two forces into a single resultant force and considering one axis of the well at right angles to this pressure.

11.3. Corrections necessary because of variation of the fundamental assumptions (4) and (5) (see para 3) have been thoroughly discussed by Terzaghi and may be neglected in those slab cases where the allowable bearing power of the soil is less than half of the ultimate bearing capacity of the soil. Usually a factor of safety of 3 is specified in calculating allowable bearing power of soil.

11.4. Doubts may be raised as to what will actually happen when plastic flow occurs and the well is allowed to tilt beyond state of elastic equilibrum of sand. The active pressure of sand may not be sufficient to bring it back to the original position until the well is again acted upon by a force from the opposite direction. The designer should necessarily find out if such type of deflection, which may be of permanent nature, should be allowed.

11.5. For designs of wells resting on cohesive soils, the present formulae for design will not hold good and it will be necessary to deduce a fresh set of formulae on the same line. The Authors desire to present a solution of that problem at a later date.

11.6. It has already been said at the outset that it has been the endeavour of the Authors to present the problem with a mathematical approach but obviously there are many limitations. No laboratory experiments have been undertaken to verify the results. The Authors hope that the subject will be given more attention by the Design Engineers of India and experiments carried out to correlate the theoretical values with the experimental results.

ACKNOWLEDGMENT

The Authors wish to thank Shri K.N. Basu, Superintending Engineer, Roads, for the encouragement given by him in preparing this Paper.

REFERENCES

1. Terzaghi, K—"Theoretical Soil Mechanics".
2. Pender, E.B.—"The Lateral Support Afforded to Piers Founded on Sand"—Vol. 19, No. 7, the Journal of the Institution of Engineers, Australia.
3. Terzaghi, K—"Evaluation of Co-efficients of Subgrade Reaction"—Geotechnique, December, 1955.

Information Section

1. SOME PRELIMINARY TRIALS FOR THE IMPROVEMENT OF WOOD TAR FOR USE IN LOW COST ROAD CONSTRUCTION
2. STRENGTH OF AXIALLY LOADED STEEL COLUMNS ENCASED IN CONCRETE
3. A RESUME ON LIME STABILIZATION WITH PARTICULAR REFERENCE TO THE EFFECTS OF LIME ON THE PHYSICS AND CHEMISTRY OF SOILS
4. SOME FIELD EXPERIMENTS ON BITUMINOUS ROAD CONSTRUCTION

"SOME PRELIMINARY TRIALS FOR THE IMPROVEMENT OF WOOD TAR FOR USE IN LOW COST ROAD CONSTRUCTION"

By

Dr. H.L. Uppal & L.R. Chadda*

CONTENTS

			Page
1.	Introduction	...	187
2.	Use of Wood Tar in Road Construction	...	188
3.	Laboratory Investigation	...	188
4.	Field Application	...	191
5.	Cost of Treated Wood Tar	...	192

SYNOPSIS

The note gives the composition, availability of wood tar from various types of wood, and how this by-product can be blended and used for checking the rise of moisture through capillary action from the subgrade in areas of high water table.

1. INTRODUCTION

Wood tar, a by-product in the manufacture of chemicals by the destructive distillation of both hard and soft wood, is available in large quantity in India.

Hard woods mostly comprise beach, oak, poplar, ash, willow, chestnut, etc., and soft woods that of pine, fir, cedar, hemlock, cypress and larch. The distillation of hard wood primarily aims at the recovery of methyl alcohol and acetates, but tar is obtained as a by-product, whereas the distillation of soft wood is mainly directed to the recovery of turpentine, wood-tar oils, tar and charcoal.

The crude products, of the distillation of hard wood can be grouped into four classes :

1. Non-condensable gases ... 20 – 30 per cent

* Central Road Research Institute, New Delhi,

2. Aqueous distillate (crude pryo-ligneous acid) ... 30 – 50 per cent
3. Wood-tar oils, and wood tar ... 5 – 20 per cent
4. Charcoal ... 20 – 45 per cent

The yield of tar from hard wood varies between 5 – 20 per cent whereas the yield of tar from soft wood is almost three times.

The tar obtained from wood distillation is regarded unsuitable for surface dressing of roads on account of the presence of about 10 – 15 per cent of water and another 5 – 10 per cent of light oils.

Wood tars are characterised by their extreme susceptibility to changes of temperature and oxidation on exposure to weather. However, these can be improved by fluxing with cashewnut shell oil, though the treatment might be uneconomical.

After the removal of water, the tar thus obtained is mostly used for:

1. Preserving of wood
2. Burning as a fuel
3. Recovery of different oil fractions.

2. USE OF WOOD TAR IN ROAD CONSTRUCTION

Some preliminary investigations were carried out to explore the possibility of using wood tar as a water-proofing material to check the rise of moisture through capillarity from subgrades and also to find out its utility as a soil stabilizer.

Some successful laboratory experiments on the stabilization of sand with tar treated with chemicals showed that the prescribed 'Hubbard' stability limits can be attained by stabilizing sand with about 6 per cent of treated tar. This experiment was carried out for application in a very dry area, as such no tests on the water-resisting quality of the treated sand were carried out.

3. LABORATORY INVESTIGATION

The treated tar used in the above experiment when tried for water-proofing of soils did not give encouraging results. Fresh experiments were taken up to improve the water-repellent qualities of the tar by treatment with chemicals. After trying a number of organic solvents, alcohol which is readily miscible with wood tar at

room temperature, was found to be most effective. Further trials were made to see as to what extent alcohol could be diluted with water without affecting solubility, it was found that the solvent with 75 per cent alcohol content was good and that only 20 per cent of the diluted alcohol was required for workable consistency. The tar dissolved in the 20 per cent diluted alcohol when mixed with soil at the rate of 5 per cent by weight did not impart enough resistance to soil against softening action of water. Further experiments were, therefore, taken up to improve water repellent qualities of the tar by use of chemicals such as rosin and stearate. As a result of trials, suitable proportions of the final mixture were found to be 80 per cent wood tar, and 20 per cent diluted alcohol containing small concentrations of rosin and stearate.

Selection of Soils

The laboratory experiments were carried out on a soil typical of alluvial plains. The physical characteristics of the soil are given below:

Liquid limit	...	32.5 per cent
Plasticity index	...	14.0
Sand content	...	15 per cent

The following aspects of the problem were studied:

(1) Effect of addition of treated tar on C.B.R. of soils.

(2) Effectiveness in checking up capillarity as a result of surface painting.

(1) Effect on C.B.R.

The effect of adding increasing percentage of treated wood tar was tried and the results are given in Table 1.

TABLE 1

Serial No.	Percentage of treated wood tar	C.B.R. (soaked)	Density lb per cu. ft.	Percentage of moisture absorption
1	0	2.3	114.6	19.67
2	1.0	3.3	116.6	15.18
3	2.0	4.0	105.8	15.88
4	3.0	3.5	105.7	14.80

It will appear from Table 1 that there is no significant increase in the C.B.R. value of soil to which increasing percentage of treated wood tar was added. The data further shows that due to rapid decrease in the dry bulk density with increasing percentage of treated wood tar, the resultant C.B.R. does not improve even when moisture absorption decreases. Similar experiments tried on the stabilization of soils with bitumen emulsion had shown that for all intents and purposes there was no increase in the C.B.R. values of alluvial soils. In one of the particular projects, the use of emulsion manufactured in the country was tried to improve the C.B.R. value of the base course in waterlogged areas, but it was found that even with fairly high concentrations, there was no marked improvement.

(2) Checking Capillary Rise of Moisture

There are two ways of maintaining the strength of soils, (i) by stabilization with bituminous material and thereby making it resistant to softening effect of water, and (ii) by stopping the capillary moisture from moving into the bases. Since stabilization of soils with treated wood tar was not found successful, further experiments were carried out to find out how subgrade painted with wood tar could check the movement of moisture.

Earlier[3] study conducted at Karnal Laboratory showed that use of shell primer No. 1 (Bitumen 80/100 - 50 parts and diesel oil - 50 parts), can be satisfactorily employed to arrest the upward movement of moisture. Laboratory studies had shown that primer at the rate of 40 lb per 100 sq. ft., in two separate coats could completely check the rise of capillary moisture through subgrade soil even when the subgrade contained 0.5 per cent sodium sulphate. Similar trials were carried out with treated wood-tar as per details given below:

Experimental Technique

Laboratory experiments were conducted in transparent plastic jars of size $4\frac{1}{2}$ in. $\times$ 6 in. having perforated bottom. Soil with a P.I. 11.2 and sand content 30 per cent was compacted at the bottom of the jar upto 2 in. thickness to a density of 1.7. The soil surface was allowed to dry and then painted with different quantities of wood-tar mixture at the rate of 15, 20, 25 and 30 lb per 100 sq. ft., respectively. The painted surfaces were allowed to dry in the sun for 24 hours to permit the penetration of wood tar. Over the painted surface another 3 in. thick layer of soil having P.I. = 8 and sand = 48 per cent conforming to base course specification was compacted with density 1.7 and then allowed to dry. In two sets of experiments, the subgrade soil (bottom layer) was mixed with 1.0 per cent and 2.5 per cent sodium sulphate to accelerate the puncturing of upper soil layer under capillary rise of moisture. All the plastic jars were kept in 2 in. deep water, and the rise of moisture resulting in the wetting of the surface was noted visually. After a lapse of 12 months, the observations given in Table 2 were recorded.

TABLE 2

Checking capillarity in subgrade soil

S. No.	Rate of application of treated wood-tar mixture	Per cent sodium sulphate	Time of rise of moisture
	lb per 100 sq. ft.		
1.	15	Nil	No rise of moisture in 12 months.
2.	20	Nil	,,
3.	25	Nil	,,
4.	30	Nil	,,
5.	15	1.0	,,
6.	20	1.0	,,
7.	25	1.0	,,
8.	30	1.0	,,
9.	15	2.5	,,
10.	20	2.5	,,
11.	25	2.5	,,
12.	30	2.5	,,
13.	Nil	Nil	Water rises in 20 hours.

4. FIELD APPLICATION

In actual field the painting of the base and sides will be carried out as per procedure given below:

A trench of desired width shall be dug to a depth, equivalent to the thickness required for the CBR value of the subgrade under soaked condition.

The subgrade shall then be rolled and brought to a proper camber. The sides of the trench shall also be pressed to avoid any loose material. The treated wood-tar shall then be painted with a sprayer both on the base and sides at the rate of 20 lb per 100 sq. ft.

If the thickness of the crust is going to be partly built up in soil, then it has to be ensured that the finished soil crust shall also be painted to avoid the moisture from the top penetrating into the consolidated soil. This will form a sausage to enclose soil to be kept dry to maintain high strength.

It will be seen as a result of laboratory experiments that painting the subgrade with treated wood tar at the rate of 20 lb per 100 sq. ft. is as effective in checking up capillarity as 40 lb of Shell Primer No. I.

Since no confirmatory results on the effectiveness of Shell Primer are available, it is too early to say how far treated wood tar is also effective. There are many unforeseen factors which play part in actual field trials which cannot be anticipated in the laboratory. It is, therefore, necessary to lay actual field trials on the procedure mentioned above before final recommendations can be made.

5. COST OF TREATED WOOD TAR

The cost of manufacturing one ton of treated wood tar has been worked out as under:

		Rs
(i)	Cost of one ton wood tar in 45-gallon drums	140.00
(ii)	Cost of methylated spirit 33 gallons at Rs 5 per gallon	165.00
(iii)	Cost of stearate and rosin	55.00
(iv)	Labour charges	10.00
	Cost of 2,720 lb	370.00
	Cost per ton (2,240 lb)	305.00

As against Rs 305 per ton of treated wood tar, the cost of Shell Primer No. I recommended earlier is Rs 400 per ton.

Since the rate of application of Shell Primer is double that of wood tar, the comparative cost on the basis of treatment with wood tar will be about 40 per cent that of Shell Primer.

REFERENCES

1. "Asphalts and Allied Substances"—Vol. I by H. Abraham, D. Van.—Nostrand Company Inc., New York.
2. "Sand Stabilization with Wood Tar"—Unpublished work of C.R.R.I.
3. Mehra, S. R., Uppal H. L. and Chadda L. R.—"Some Laboratory Experiments on Use of Primers in Preventing Capillary Rise—I.R.C. Journal, Vol. XIX, Part 3, 1955.

"STRENGTH OF AXIALLY LOADED STEEL COLUMNS ENCASED IN CONCRETE"

By

P. C. VARGHESE

SYNOPSIS

This Paper gives the results of an experimental investigation on strength of axially loaded encased steel columns. According to the present Indian specification, the concrete in centrally loaded encased steel columns is permitted to assist the steel section in carrying the load only if they are reinforced with spiral binders. In the light of experimental investigations, this provision seems to be very conservative. Presence of binders has very little effect on the total load carrying capacity of these columns. However, they affect the type of failure considerably. With binders the failure is less brittle whereas without binders a splitting type of failure takes place. Experiments indicate that rectangular binders, which are commonly used in actual construction work, will also give ample protection against these brittle failures.

1. INTRODUCTION

1.1. The idea of using a large metal core encased in concrete as columns was first conceived by Von Emperger as early as 1905. These columns consisted of cast iron metal core covered with concrete which was laterally reinforced with spiral binders. Such types of columns were extensively used in building construction in Europe and U.S.A. some fifty years ago. Comprehensive tests on these columns were made in those days and the columns were designed to carry much heavier loads than the corresponding metal columns not encased.

1.2. Tests on encased *steel columns* were carried out by Talbot and Lord as early as in 1912 to investigate the value of concrete as reinforcement for structural steel columns[2]. Fabricated steel (open hearth) columns 2 ft to 9 ft 4 inches long were tested as plain steel columns, as core type columns (i.e., with the space inside the fabricated steel column filled with concrete) and as spiral columns (i.e., with concrete cover outside the fabricated steel column and the concrete reinforced laterally by spirals). They found that all these columns had qualities of good structural members. Further tests were made on encased steel columns with rectangular binders by U.S. Bureau of Standards in connection with the design of George

Washington Bridge[3] and by Oscar Faber in 1955[4]. These tests showed that the encased steel columns, with rectangular binders as encasement reinforcements, were safe to use in practice and that their carrying capacity was much higher than the corresponding steel columns.

2. PRESENT-DAY PRACTICE

2.1. Till recently codes of practice recognised only the stiffening effect of such encasement and the concrete was not considered as carrying any portion of the load. Modern codes however, recognise the load carrying capacity of the concrete encasement and allow a certain portion of the load to be carried by the concrete. However, opinions differ widely on the question of the amount of allowable load.

2.2. The British Code (B.S. 449—1959, clause 30) stipulates that for symmetrical sections, effectively reinforced with 3/16 in. dia. wires in the form of *stirrups or binders,* the concrete cover upto a maximum of 3 in. may be assumed to assist in carrying the load. This concrete may be assumed to carry a stress equal to the allowable stress in steel divided by 30. The safety of these provisions has recently been verified by a series of tests by the Building Research Station, Watford[5].

2.3. Similarly, the revised Indian Standard Specification (I.S. 456—1957, clause 6.33.15 with erratum No. 4) specifies that the strength of steel or cast iron columns encased in *spirally* reinforced concrete may be calculated from the formula

$$P = AC + A_c C_1 + f_m A_m,$$

where A_m = Area of steel or C.I. core

A_c = Area of cross-section of bar reinforcement

A = Net area of concrete section

C = Permissible stress in concrete in direct compression.

C_1 = Permissible compressive stress for column bars

f_m = Allowable unit stress in metal core.

2.4. Both the above codes have their limitations. Thus while in B.S. Code the strength of the column is independent of the strength of concrete used, the carrying capacity of the columns in Indian Code is based on ultimate load which is rightly a function of the strength of the concrete. However, B.S. Code allows increase in strength for all columns used in practice, as only rectangular binders are generally used as lateral reinforcements in encased columns.

Photo 1

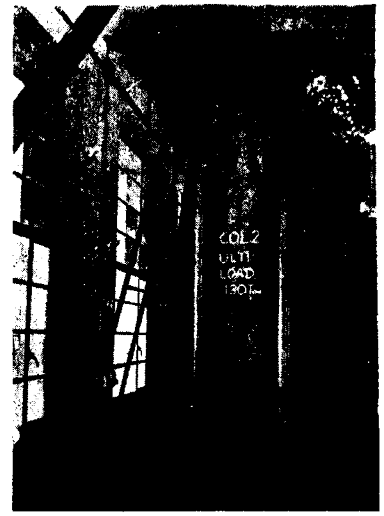


Photo 2

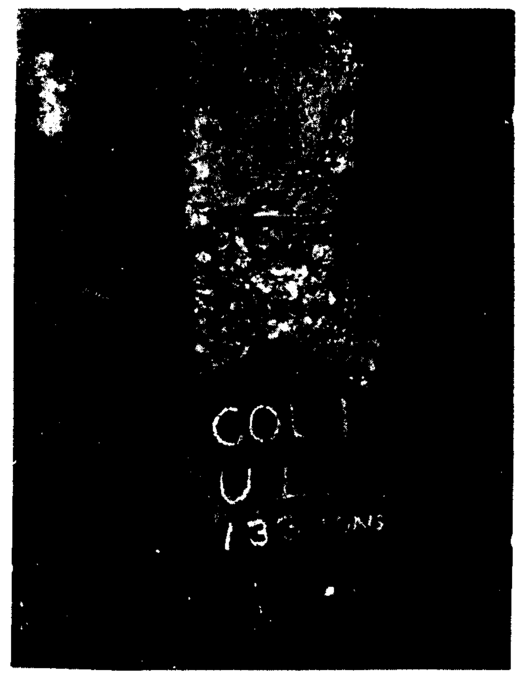


Photo 3

Photo 4

Photo 5

Indian code allows the increase only in columns provided with special spiral reinforcements.

3. EXPERIMENTAL INVESTIGATION

3.1. With a view to assess the true load bearing capacity of these columns and also to know the function of binders in these columns some tests were made at the Indian Institute of Technology, Kharagpur, on encased centrally loaded columns. Photos 1 and 2 show the general set up for the two tests and Photos 3 to 5 show features of failure obtained in these columns. Table 1 gives the results of tests.

3.2. Two types of test specimens were made, one set with simple steel joists encased in concrete without encasement reinforcements and the other with rectangular binders and longitudinal rods as encasement reinforcements. Wherever provided, these encasement reinforcements consisted of 4 nos. 1/4 in. diam. vertical rods with 1/4 in. diam. rectangular binders at 6 in. centres. As in Oscar Fabers' test most of these columns with uniform rectangular sections failed at its ends. It was, therefore, thought worth while to test similar columns with flared ends, Photo 4.

3.3. During the test the surface strains in the concrete were measured at different elevations on all four sides by demountable strain gauge of 8 in. gauge length. In a few tests, stress in steel was also measured by means of electrical resistance gauge attached to the steel section and embedded in concrete. The latter readings showed that there was no tendency for slip between the steel and encasement during the test.

4. DISCUSSION OF RESULTS

4.1. It is well-known that mild steel reaches its yield at a strain of about 0.0015. Also, the failure strain of plain concrete in compression is of the order of 0.002. This failure strain may be increased to a much greater value with provision of closely spaced rectangular, circular or helical binders[6]. Rectangular binders by their very geometry will not be as efficient as helical or circular binders in increasing this strain at failure, but it can easily make the concrete to sustain *a safe failure strain value* much greater than the yield strain of steel. Thus, if concrete and steel are stressed together in compression, and if there is no bond slip between them, it should quite naturally be expected that, if loaded to failure, both steel and concrete will be stressed to its capacity. Thus, as in reinforced concrete columns, the ultimate load carrying capacity should be able to be expressed as

$$P = As\, fy + As'\, fy' + A_c \propto C_u$$

where As = Area of encased steel column

fy = Yield strength of encased steel

As' = Area of longitudinal steel rods

fy' = Yield strength of steel rods

Ac = Net area of concrete

$\propto C_u$ = Average stress developed in concrete.

4.2. Examination of the test result shows that the value of $\propto$ varied from 0.4 to 0.7. There was no noticeable difference between the strength of columns of uniform section and columns with flared ends, except that the failure in the latter case occurred away from the ends. Also, there was no appreciable difference between the ultimate strength values of encased columns with and without rectangular binders. However, in general, the provision of binders produced higher concrete strains at failure; also the failure was more localised and less brittle with these binders. Without binders a splitting type of failure as shown in Photo 5 was obtained in the specimens.

5. CONCLUSIONS

5.1. This investigation clearly points out that the provisions of rectangular binders as encasement reinforcement ensure prevention of brittle failures in such columns and enables the structure to attain its ultimate load safely. As in reinforced concrete columns provision of helical binders may further increase the strain at failure of these columns but is not necessary for developing its full strength.

5.2. The ultimate strength of such columns can be expressed by the formula—

$$P = As\, fy + As'\, fy' + Ac \propto C_u$$

and this may be used with suitable load factor in design. The value of $\propto$, for the range of columns tested, varied from 0.4 to 0.7.

5.3. In the light of these experimental evidences the present provision in the Indian Code of Practice, of allowing increase in strength only with the provision of *special helical binders* is conservative.

ACKNOWLEDGMENT

The investigation described in this Paper was undertaken at the Indian Institute of Technology, Kharagpur, with financial assistance from the Council of Scientific and Industrial Research. The help and encouragement received by the Author from Dr. S. R. Sen Gupta, Director, Indian Institute of Technology, in this work is gratefully acknowledged.

REFERENCES

1. John Tucker—" Reinforced Concrete Columns "—Proc. Am: Soc. Civil Engr.—1923.
2. Talbot, A. N. and Lord, A. R—" Tests on Columns—An Investigation of the Value of Concrete as Reinforcement for Structural Steel Columns "—Ill. Univ. Bull. No. 56, March, 1912.
3. Stang and Whitmore—" Research Paper R. P. 831 "—National Bureau of Standards, U.S. Dept. of Commerce.
4. Oscar Faber—" Savings to be Effected by a More Rational Design of Encased Stanchions as a Result of Recent Full Size Tests "—Str. Eng., March 56. April, 57.
5. Thomas, F. G.—" Strength of Encased Steel Stanchions "—Engineering Vol. 180—pp 789 (1955).
6. Baker, A. L. L.—" The Ultimate Load Theory Applied to the Design of Reinforced and Prestressed Concrete Frames "—Concrete Publication Ltd. (page 28).

TABLE 1

Summary of tests on axially loaded short encased columns

Col. No.	Description	$\frac{L}{b}$	Ultimate load by test P	P=As fy ult: load as plain steel col.	Value of ∝ in P=As fy+∝ $C_u A_c$	Max. strain measured before failure.
			Tons	*Tons		$(10^{-5})x$
1	Plain steel R.S.J. 5 in.×3 in., 84 inches in height		40	60.96	(buckling)	
2	5 in. × 3 in. R.S.J. encased to 9 in. × 7 in. with binders and c (encasement reinf. :) 84 in.	12	130	64.10	0.477	141
3	Same as in No. 2	12	133	64.10	0.498	140
4	7 in. × 4 in. R.S.J. encased to 11 in. × 8 in. without encasement reinforcement : 12 in.	1.5	157	99.75	0.560	475
5	Same as in No. 4 with encasement reinforcement	1.5	148	102.88	0.467	660
6	4 in. × 1¾ in. R.S.J. encased to 6 in. × 3¾ in. without encasement reinforcement : 12 in. height	3.2	62.5	29.40	0.514	194
7	Same as in No. 6 with encasement reinforcement	3.2	64.0	32.54	0.490	548
8	4 in. × 1¾ in. R.S.J. encased to 6 in. × 3¾ in. without encasement reinforcement 54 in. height	14.4	53.6	29.40	0.386	140
9	Same as in No. 8 with encasement reinforcement : (C_{2-0})	14.4	58.0	32.54	0.445	144
10	4 in. × 1¾ in. R.S.J. encased to 9 in. × 3¾ in. without encasement reinforcement 54 in. height	14.4	75.9	29.40	0.529	157
11	Same as in No. 10 with encasement reinforcement : (C_{3-0})	14.4	90.2	32.54	0.631	215
12	4 in. × 1¾ in. R.S.J. encased to 8 in. × 6 in. with 6 in. × 4 in. binders tied to 4 Nos. long : rods and ends of column flared to 8 in. × 8 in. = 50 in. height	8.33	129.5	32.54	0.683	329
13	Same as in No. 12 with plane ends height = 48 in.	8.0	117.6	32.54	0.600	287
14	4 in. × 1¾ in. R.S.J. encased to 8 in. × 6 in. with 7 in. × 5 in. binders tied to 4 Nos. long : rods. Ends of col. flared to 8 in. × 8 in., height = 50 in.	8.33	118.3	32.54	0.675	32[illegible]
15	Same as in No. 14 with plane ends height = 48 in.	8.0	117.2	32.54	0.660	387
16	4 in. × 1¾ in. R.S.J. encased to 8 in. × 6 in. without encasement reinforcement : Ends of column flared to 8 in. × 8 in., height = 50 in.	8.33	116.1	29.40	0.747	2[illegible]
17	Same as in No. 16 with plane ends, height = 48 in.	8.0	123.7	29.40	0.740	262

* These include load carried by the encasement reinforcement where provided.

" A RESUME ON LIME STABILIZATION WITH PARTICULAR REFERENCE TO THE EFFECTS OF LIME ON THE PHYSICS AND CHEMISTRY OF SOILS "

By

DR. H. L. UPPAL* & R. M. PALIT*

SYNOPSIS

As a result of research work carried out in different parts of the world, during the last decade or so, the stabilization of soils with lime has shown a very promising future. Lime soil seems to have now come to stay as a material of construction with comparatively low strength.

There is, however, a lot of controversy as to the nature of actual reaction that takes place when lime is added to clays. A resume of the work already done is presented in this Paper, to serve as a guide for further work in this field.

1. INTRODUCTION

It is known that in the second century, Romans used lime in building industry by mixing it with volcanic sand, called pozzolana named after the place where it was found.

Use of lime continued extensively right up to twentieth century when replaced to a large extent by cement, a much stronger and standardised material.

Beyond extensive use of lime in building industry, there is no literature available to show that lime had also been used for improving earthen roads.

The first attempt to improve soil with lime for roads was perhaps made in 1924, in the State of Missouri, as an experiment on the construction of a road using hydrated lime[1]. In spite of some initial success in the stabilization of soil with lime, no serious attempt was made to extend the technique to a large scale construction till the second world war when gravel stabilized with lime was used in the construction of runways and taxiways in Texas.

Laboratory as well as field experiments were also taken up by the Texas Highways Department and by 1953 about 250 miles of roads had been constructed in Texas using soil stabilized with lime[2].

* Central Road Research Institute, New Delhi,

In 1954, the National Lime Association of America issued a booklet describing the method of determining lime required for stabilization of soil in the laboratory and the technique of constructing stabilized soil roads[3].

Clare and Cruchley[4] have reported that considerable work on the stabilization of soil with lime has been carried out besides U.S.A., in Russia by Okhtina, Bykovskii, Volkov, Gelmer, Zasobin, Panteleev, Groditskaya, Ipatova. According to the work done in Russia, use of unslaked lime for stabilising soil has been found to be more economical and effective than slaked lime, and that the strength of treated soil keeps on increasing over a period of few months depending on the type of soil and the concentration of lime which varies from 1–10 per cent. The addition of lime makes the clayey soils friable, and thus helps in increasing construction speed considerably. Beside U.S.A. and Russia, considerable work on the stabilization of soil with lime has also been done in the U.K., India and Israel, but it has mostly been confined to the laboratory study.

2. STABILIZATION OF SOILS WITH MIXTURES OF LIME AND OTHER COMPOUNDS

2.1. Lime and cement

Some of the soils, which on account of their high colloidal contents require very high percentage of cement, can be economically stabilized, if first treated with lime to reduce plasticity, and then stabilized with comparatively smaller concentration of cement. This process has been found very satisfactory for stabilization of black cotton soils in India[5]. Similar results have also been reported by other workers in different parts of the world [6,7].

2.2. Lime with Flyash

In recent years the use of lime and flyash for the stabilization of soil has received considerable attention. Flyash is the grey dust like ash and is the residue from burnt powdered coal. The ash is similar to the volcanic ash, used in early Roman constructions as a pozzolanic material, that is, material not in itself a cement, but it reacts with lime and water to form a weak cement. While the study of lime-flyash-soil treatment has not extended to a point where full information is available, the following conclusions may be derived from existing literature [8 to 13]:

(i) The addition of small amounts of hydrated lime and flyash develops concrete like composition of high strength at relatively early ages. When compacted with optimum moisture, unconfined compressive strength of the order of 1,000 lb per sq. in. in 28 days is developed. When cured at higher temperature which accelerates the pozzolanic reaction of lime and flyash, the compressive strength obtained in 7 days is of the order of 350 to 1,400 lb per sq. in. depending upon the type of the soil.

(ii) The weathering resistance of the composition appears to be exceptionally good. The test for wetting and drying, and freezing and thawing show excellent resistance to deterioration after 12 cycles of treatment.

(iii) With addition of lime and flyash maximum strength is obtained when compacted at a moisture content equal to or slightly less than the optimum.

(iv) Strength increases with the degree of pulverisation.

(v) Strength increases with increase in curing time.

(vi) Elevated temperature curing greatly increases strength.

(vii) Specimen sealed from contact with atmosphere gives greater strength after six months or more than unsealed specimen.

(viii) Small percentage of calcium chloride gives substantial increase in strength of stabilized clayey and sandy soil.

(ix) Addition of lime-flyash improves the consistency limits and shrinkage properties of soil.

2.3. Mixture of Lime and Magnesium oxide

Lu[14] and his co-workers found that lime cementation of soil at ordinary temperature is best when lime is mixed with equal molar part of MgO. Highest strength is obtained, when dolomite quick lime is used and allowed to hydrate with soil.

The relation between strength and Ca : Mg ratio is illustrated in Fig. 1. With monohydrate lime the optimum Ca : Mg ratio approximates that of pure dolomite lime.

2.4. Lime with bituminous binder

Experiments have been carried out in the Road Research Laboratory, U.K.[15], and Central Road Research Institute, India[16], on the use of lime and wood tar for stabilizing soil. Texas Highways also has suggested that clays of the montmorillonite or bentonite

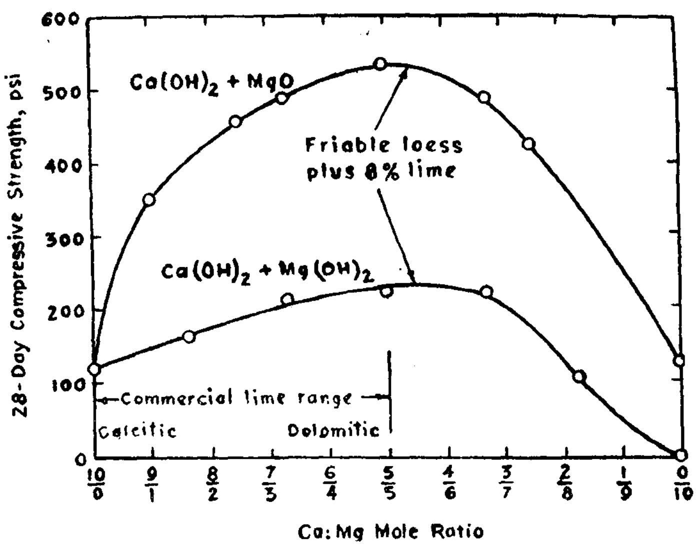


Fig. 1. **Effect of Ca : Mg ratio on immersed strength of friable loess with 8 per cent synthetic Hydrated Lime**

type, can be best stabilized with lime and bitumen. The stabilization is perhaps due to the waterproofing property of bitumen.

It has been found out that the effect of lime is improved to a considerable degree by adding wood tar, a by-product in the destructive distillation of wood.

3. PHYSICS AND CHEMISTRY OF LIME SOIL REACTION

It has been seen as a result of study of literature reviewed in the previous pages, that some of the soils react very favourably with lime in creating resistance to softening action of water. A number of research workers had been engaged to find out the type of reaction that takes place by adding lime to soils. Some of the theories advanced by various research workers explaining the reactions are discussed in the following pages. This has been divided into two parts (i) physical properties (ii) chemical properties.

3.1. Effect of lime on physical properties of soils

3.1.1. Liquid limit

The effect of addition of lime on the liquid limit of different types of soils has been studied by research workers in various parts of the world. The results are conflicting in as much as, some have observed rise in liquid limit values, whereas others have found loss in values. Research carried out in India[5,16] with black cotton soil has shown that liquid limit of the soil decreases with the increase in the concentration of lime. Similar observations have also been made in U.S.A. by Spangler and Patel[17] on heavy clayey soils. But Clare and Cruchley[4] in U.K. and Zietlen and Zolkov[18] in Israel report a complex reaction. According to the work done in the U.K.[4], the immediate effect of the addition of small concentration of lime such as 1 per cent raises the liquid limit of the soil considerably, but further addition of lime upto 10 per cent steadily reduces this value, though at no stage lower than the liquid limit of untreated soil.

According to the work done in Israel[18], liquid limit of soil decreases with the increasing concentration of lime when tested immediately, but with lapse of time over a year or so, the liquid limit has a tendency to rise.

3.1.2. Plastic limit

In so far as the effect of addition of lime on the plastic limit of soils is concerned, there is almost a general agreement that plastic limit always increases with addition of lime, whether tested immediately or after lapse of time.

3.1.3. Plasticity index

Plasticity index generally falls as a result of addition of lime. In the case of black cotton soil, the reduction in plasticity index is brought about by comparatively rapid decrease in liquid limit than corresponding rise in plastic limit with increasing percentage of lime. It was also observed that with 5 per cent addition of lime to black cotton soil the plasticity index falls from 45 to about 5. Further addition of lime upto 10 per cent may render the soil non-plastic, though from practical point of view it may not be desirable. The data available from research work carried out in the U.K[4]. and Israel shows that even with 10 per cent addition of lime the fall in plasticity index of clayey soils is not very marked even when lime is allowed to react for a year or so. There is also an indication that maximum reduction in plasticity index is recorded immediately after the addition of lime, but with lapse of time there is some increase in the plasticity index. Research work in U.S.A.[19] on silty soils shows that with addition of lime there is an increase of plasticity index.

3.2. Other properties of soils

Results of research work on other physical properties of soils such as shrinkage limit, shrinkage ratio, volumetric shrinkage, swelling characteristics, etc., show that after treatment with lime, the soil improves resulting in increase of shrinkage limit.

3.3. California bearing ratio

A clayey soil has a very low C.B.R. in saturated condition, as such is considered unsuitable for use even as a subgrade. Research work carried out on clayey soil, such as black cotton soil, has shown that as high as 80 per cent soaked C.B.R. can be obtained in the laboratory with only 5 per cent addition of lime[16]. Depending upon the effectiveness of mixing and degree of pulverisation of soil that can be obtained in the field, it should be possible to obtain at least 50 per cent of laboratory C.B.R. in actual practice which will be a great improvement on the strength of natural soil.

The work carried out in Israel and India shows that soaked C.B.R. of soil lime mixtures increases with the increase in percentage of lime. The highest value attainable depends on the nature of soil.

3.4. Unconfined compressive strength

Most of the clayey soils which have ridiculously low strength in soaked condition improve considerably by treatment with lime. It has been shown as a result of research work carried out in India on black cotton soil that the compressive strength can go as high as 150—200 lb per sq. in. after 7 days curing. It has also been noticed that further addition of lime beyond certain limits causes reduction in strength. Work done by Fuller & Dabny[20] has shown that the ultimate compressive strength of lime treated soil, rises progressively at least for 300 days resulting in an ultimate compressive strength of 780 lb per sq. in. Recent research work has shown that higher compressive strength can be obtained if instead of calcium lime a mixture of calcium and magnesium lime in equal proportion is used.

4. CHEMISTRY OF REACTION

Some of the theories put forward by various research workers to explain the reaction that takes place when lime is added to clay are discussed below.

4.1. Flocculation

The agricultural chemists maintain that there is no direct effect of lime on soil structure. It is, however, recognised that lime promotes better structure through its indirect effect on organic matter production and microbiological activity. This theory of organic complex formation is, however, not supported by work done in soil mechanics where even a small percentage of lime when added to soil and subsequently compacted, results in the development of a hard mass even when there is no organic matter present.

4.2. Base exchange

Many properties of soil-water system such as plasticity index, sensitivity, permeability and swelling are influenced by the nature of exchangable cations. Thus all other constants being equal a soil carrying Na^+ as exchangable ion is likely to have different plasticity index, swelling pressure and other properties than a soil carrying Ca^{++} ion. An exchange of a relatively small amount of Na^+ by Ca^{++} may result in a major change in the physical property of the clay.

The theory of base exchange has been advanced to explain the beneficial effect of lime on soil. It is argued that calcium ion of lime displaces the exchangable Na ion of the clay, and thereby flocculates the soil, which is comparatively more stable. On the basis of this theory the stabilization of black cotton soil with lime cannot be explained as the soil is already Ca^{++} soil, and, therefore, no further base exchange with lime can take place.

It is also observed that every salt of Ca cannot bring about the beneficial effect, e.g., even though $CaCl_2$ brings about base exchange, it has no stabilizing effect.

4.3. Formation of calcium carbonate

Goldberg and Klein[21] after carrying out studies on X-ray diffraction and differential thermal analysis of two clays (portervile clay containing 50 to 60 per cent montmorillonite and bentonite) concludes that most of calcium hydroxide is converted into calcium carbonate, and no free calcium hydroxide remains after some time. Their studies do not disclose detectable amounts af hydrosilicate of lime. Chemical analysis of treated portervile clay shows progressively higher values of calcium carbonate when treated with increasing concentration of calcium hydroxide content as a result of partial carbonation of calcium hydroxide. Calcium carbonate was also found in the lime treated bentonite which originally contained no calcium carbonate.

However, Clare and Cruchley[4] have found that the formation of carbonate is not essential for imparting strength to soil stabilized with lime. They made samples of soil stabilized with lime and gave a heavy coating of paraffin wax to prevent access of air so that no CO_2 from air can react with lime. It has been found that even then, the soil develops strength and, therefore, contradicts the theory of the formation of carbonate to impart strength to lime stabilized soil. It could, however, be argued that the air entrapped in the voids of the blocks contains enough CO_2 to convert Ca $(OH)_2$ into $CaCO_3$. To avoid this argument, work was conducted in the Central Road Research Institute by curing the lime treated blocks in vacuum desiccator, which removed the air completely. It was noticed that after curing for a week or so the blocks developed the same strength as those cured in air, thereby disproving the theory of carbonate formation.

4.4. Calcium Hydroxide

One of the theories put forward is, that the increase in strength of soil stabilized with Ca $(OH)_2$ could also be due to the hexagonal crystals of Ca $(OH)_2$ which can be formed at temperature of 28° to 30°C and under reduced pressure conditions which are met with under actual field condition. It was shown as a result of experiments carried out in the U.K. that even when required temperature and moisture conditions for the crystalisation were not provided, there was still improvement in the soil strength thereby challenging the application of this theory.

4.5. Formation of Cementitious Calcium Silicate

Now-a-days it is generally agreed that the most possible mechanism involving the stabilization of soil by lime is the formation of cementitious hydrated calcium silicate. "A number of hydrated calcium silicates have been reported in nature and some have been prepared artificially. The products obtained by the wet process usually vary in composition and have the character of colloids. Towards the end of eighteenth century Godolin precipitated alkali silicate by calcium salt, and by treating hydrated silica with calcium hydroxide. Fuch stated that hydrated silica when added to lime water, precipitated a white calcium silicate. Heldt observed that silicic acid mixed with milk of lime forms a hard mass. Landin stated that gelatinous and dialysed silicic acid acts on lime water forming various hydrated calcium silicate which plays a role in the setting of hydraulic cement. He presented the composition of the precipitate as 4 CaO 3 SiO_4 or 2 CaO, 3 SiO_2. This subject was studied by Lechaterler who demonstrated that products with a composition between CaO, 10 SiO_2 and 2 CaO SiO_2 can be obtained by varying proportion of lime and silica used in their preparation[22]."

It is, therefore, quite probable that even at ordinary temperature, some reaction of lime and silica of the clay takes place in the wet condition to precipitate a cementitious calcium silicate into the soil mass.

This is further supported by the work carried out in the C.R.R.I. The development of strength in soil stabilized with Ca $(OH)_2$ depends upon the availability of silicates to form complex compounds of Ca with iron and aluminium silicates. It will be observed that when soil is burnt to a temperature, where the clay structure starts disintegrating and maximum, percentage of silicates are exposed, then the development of strength as a result of addition of Ca $(OH)_2$ is maximum.

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REFERENCES

1. McCoustland, E.J : "Lime in dirt road"—Proc. National Lime Association Convention, 1925.
2. American Road Builders Association: "Report of Committee on Lime/soil Stabilization": Tech. Bul. No. 147, 1948.
3. National Lime Association: "Lime Stabilization of Roads"—Bul. 323, 1954.
4. Claro, K.E. and Cruchley, A.E.: "Laboratory Experiments in the Stabilization of Clays with Hydrated lime"—Geo-technique, Vol. VII-No. 2, 1957.
5. Bose, S.K.: "Stabilization of certain Clayey Soils in India by Heat treatment"—J.I.R.C. Vol. XVIII-2, 1953.
6. Brooke Bradley, H.E.: "Soil Cement Roads in Worcestershire"—Surveyor 111, 1952.
7. Kerr H.L.: "Soil Cement in an Urban Area"—Surveyor 112, 1953.
8. Minick, L.J. and Miller. R.H.: "Lime Flyash Composition for Use in Highway Construction"—Proc. H.R.B., 1950.
9. Minick, L.J. and Miller, R.H.: "Lime Flyash Composition in the Highways"—Proc. H.R.B., 1951.
10. Leadaler, Norting and Myers: "Properties of Lime Flyash Soil Composition Employed in Road Const."—H.R.B. Bulletin 69.
11. Chu and Davidson: "Soil stabilisation with Lime Flyash Mixture"—H.R.B. Bulletin 108.
12. Goecker, Davidson & Chu: "Stabilisation of fine and Coarse Grained Soils with Lime Flyash mix."—H.R.B. Bull. 129.
13. Minick: "Soil Stabilisation with Lime Flyash Mixing"—H.R.B Bull. 129.
14. Lu, Davidson Hardy: "The Calcium magnesium Ratio in Soi Lime Stabilisation"—Pro. H.R.B. 1957.
15. Grainger, G.D.: "Stabilization with lime and local wood tar of a Black Cotton Soil from Southern Rhodesia"—Road Researc Laboratory Note No. RN—1778/GOG (unpublished).

16. Uppal, H.L. and Bhatia, H.S.: "Stabilisation of Black Cotton Soil for Use in Road Construction"—RR Bulletin of I.R.C., 1958.

17. Spangler, M.G. and Patel, O.H.: "Modification of Gumbotil Soil by Lime and Portland Cement Admixture"—Proc., H.R. 1949.

18. Zeitlen, J.G. and Zolkov, B.: "Lime stabilization of Fat Clay" Regional Conference (Asia) on Soil Mechanics and Foundation Engineering, 1960.

19. Woods, K.B. and Yoger E.J.: "Stabilization with Soil Lime or Calcium Chloride as an Admixture"—Proc. of conference on soil stabilization. Massachusetts Institute of Technology 1952 (1).

20. McAllister, R.W.: "Report to the Mississipi Lime Company on Lime Stabilised Highway Construction.

21. Goldberg and Klein: "Some Effects of Treating Expansive Clays with Calcium Hydroxide"—A.S.T.M. Special publication No. 142, 1952.

22. Mellor: "A Comprehensive Treatise on Inorganic and Theoretical Chemistry", Vol. VI.

"SOME FIELD EXPERIMENTS ON BITUMINOUS ROAD CONSTRUCTION"

By

D. C. CHATURVEDI*, C.E. (Hons.), D.I.C. (London), A.M.I.E. (Ind.)

&

J. C. TAYAL†, B.E.

CONTENTS

		Page
1.	Introduction	... 213
2.	Organization	... 215
3.	Field Experiments	... 215
4.	Conclusions	... 225

SYNOPSIS

The Paper discusses some problems being faced at present in the construction and maintenance of surface dressed and water-bound macadam roads.

Details of laboratory tests and field experiments carried out by U.P., P.W.D. Research Institute on Lucknow-Sultanpur road on the use of (1) brick grit, (2) precoated stone and brick grits, (3) built-up spray grout specifications using brick metal, (4) simultaneous coat of tar and bitumen in surface dressing and (5) diluted emulsion in water-bound macadam are described and the possibilities of further development of these techniques are discussed.

1. INTRODUCTION

1.1. In many parts of U.P. stone metal is expensive and, therefore, the search for suitable alternate materials has resulted in wide scale adoption of overburnt brick metal for water-bound road construction[1 & 2]. Still, for surface dressing work or premix

* Executive Engineer, U.P., P.W.D., Agra.

† Assistant Engineer, U.P., P.W.D., Lucknow.

carpet, stone grit has to be brought from long distances. This makes the cost of surface dressing or premix carpet high. Use of brick grit with and without any special treatment, precoating may be effective and economical in many situations.

1.2. Due to fast moving traffic on roads, the loss of grit from newly surface dressed roads is sometimes so much that the surface of the W.B.M. becomes bare, thus shortening the life of surface dressing. The loss of grit is moreover pronounced on the curves. Precoated tar chippings have been used in U.K. with quite encouraging results[3]. There are some stone grits which are available at economical cost but show poor adhesion to binders and are thus not favoured. By precoating these grits, adhesion can be improved and loss of grit reduced.

Precoating has another possibility, viz., in the use of softer type of grit such as brick grit, it is possible to improve the abrasion quality of brick grit and crushing strength by precoating with small quantities of tar or bitumen.

1.3. Surface painting of water-bound macadam is normally carried out when the surface is dry. In case of water-bound brick ballast, surface painting has to be done as early as possible to avoid crushing of brick ballast under traffic. It will, therefore, be better if bituminous coat can be given simultaneously with the consolidation of brick ballast. In Punjab, specifications for built-up spray grout have been tried on some roads[1], and found successful. In these specifications, consolidation of brick metal and painting are carried out simultaneously, thus obviating the necessity of bringing the surface to a dry stage, constant blinding and delaying the painting over the surface.

1.4. For surface dressing in U.P., road tar is used in first coat painting followed by second coat with bitumen. There is always some time lag between first and second coat of painting depending upon the availability of funds. This often results in the wearing out of first coat and when the second coat is given, the surface is in such a condition that the second coat painting becomes more or less a first coat, necessitating early renewal. Use of bitumen for first coat of painting is sometimes adopted when two coats cannot be done for lack of funds. However, it is considered better to provide first coat with a material which has sufficient penetrating properties followed by a second coat with a material which is resistant to wear and is durable.

If, therefore, both road tar and bitumen are used simultaneously for painting, the advantage of both the products may be obtained.

1.5. The water-bound surface after consolidation needs constant blinding (covering) in order to keep it intact under the action of traffic. This is all the more important when bituminous coat is to be given. Normally consolidation of W.B.M. is done during rainy season (July-Sept.) and it is not possible to put down bituminous coat over the surface immediately after consolidation as the surface does not become dry for considerable period. Necessity is, therefore, felt for evolving some effective method to maintain the W.B.M. surface in order.

Bituminous emulsions have been used in U.S.A. for stabilization of water-bound bases[4]. There are, therefore, possibilities of using diluted emulsion during consolidation of water-bound macadam to stabilize the crust to some extent and obviate the necessity of frequent blinding.

2. ORGANISATION

2.1. With the object of evolving suitable specifications and techniques for the solution of above mentioned problems in bituminous road construction, it was considered essential to lay down under the direct supervision and control of U.P., P.W.D., Research Station, test lengths with different specifications on the road itself for observing behaviour under actual traffic and climatic conditions.

2.2. Trial lengths concerning above problems were laid on Lucknow-Sultanpur road, Appendix I, which is a State Highway carrying fairly high traffic, Appendix II, which is proposed to be provided a modern surface.

3. FIELD EXPERIMENTS

3.1. Use of brick grit for surface painting

3.1.1. In connection with the investigations for production of high quality bricks for use in road pavement[5], a brick kiln was set up on this road. Side by side with the production of straight bricks, a good amount of misshapened overburnt bricks was also produced in the kiln. These bricks were generally broken and used as brick metal in W.B.M. construction.

The brick metal produced from some of these bricks showed low values of Los Angeles Abrasion and Aggregate Crushing thereby indicating their suitability as a road metal. It was considered that use could also be made of brick grit produced from such bricks for surface painting work. The results of laboratory tests on brick grit are given below:

Sl. No.	Brick grit used in mile	Los Angeles Abrasion (value per cent fines)	Aggregate crushing value (per cent fines)
1	22	27.0	20.4
2	29		
3	12	29.8	23.2

3.1.2. Test lengths were laid in mile 22 and mile 29 using Road Tar Grade R.T. 2 as binder and brick grit of appropriate size as gritting material. The W.B.M. surface in mile 22 was of granite stone (Jhansi) and that in mile 29 was of overburnt brick metal (obtained from the same kiln). The quantity of binder and grit used was same as is normally specified, viz., binder at 44 lb per 100 sq. ft. and grit at 4.4 cu. ft. per 100 sq. ft. The length in mile 22 was under traffic for 8-9 months before second coat of painting was given with Mexphalt 80/100 at 22 lb per 100 sq. ft. and brick grit at 2.2 cu. ft. per 100 sq. ft. The surface had been behaving fairly satisfactorily as compared to the surface where granite chips were used in adjoining stretches. The second coat of surface dressing had also been behaving satisfactorily. In mile 29, the length has been under observation for about 18 months and surface was satisfactory. Second coat with brick grit and Mexphalt 80/100 was given in May, 1960.

3.1.3. In mile 12, brick grit obtained from overburnt bricks produced in a local kiln, was used in a renewal coat with Mexphalt 80/100. The surface is still under observation and no patches have formed during the last one year since it was laid.

3.1.4. It was, however, observed in all these lengths that the quantity of binder used was slightly inadequate as the binder did not work up fully. It appears that some binder was absorbed by the grit itself. It is suggested that the quantity of binder should be increased by about 10 per cent to compensate the loss of binder due to the absorption by brick grit. This will not mean any great increase in the cost of painting as the cost of brick grit is appreciably low compared to stone grit, Appendix III.

3.2. Use of precoated grit for surface dressing

3.2.1. The procedure for precoating of chips consists of mixing the heated stone grit with 0.8 to 1.2 per cent of tar at a temperature

of 260°F. The higher temperature of mixing makes the grit non-tacky at normal application temperature and it has been found that the precoated chips can be stored for considerable time before use. The grade of Tar is not considered material for this purpose and precoating.

3.2.2. Precoating of chips with bitumen had not been tried in U.K. but it was considered by the Authors that this might also be tried where bitumen is to be used as binder. A temperature of 400°F was considered suitable for heating of grit before mixing. The heating of stone grit for the experiments was done in half-cut tar drums on the road side and mixing was done in hand-drum mixers, Photo 1. For both tar and bitumen, the quantity of binder was kept as 0.8 per cent of the weight of the stone grit. It was seen that this quantity of binder was sufficient to give a thin layer of binder over the stone chips. The precoated chips did not agglomerate on cooling and could be handled easily with shovels at the time of gritting.

3.2.3. Precoated grit for first coat painting

3.2.3.1. Test length was laid in mile 42 using precoated stone chips ½ in. size and Road Tar Grade R.T. 2 was used in first coat surface dressing over stone water-bound. The quantity of tar used was 40 lb per 100 sq. ft. against the normal 44 lb per 100 sq. ft. to save the binder used up in precoating control length using normal stone chips and tar at 44 lb per 100 sq. ft. was laid side by side. The two surfaces were kept under observation and it was seen that the whip off by traffic in case of precoated grit was much less than the portion where normal grit was used, Photo 2.

3.2.3.2. In order to account for the cost of labour involved in precoating, it was thought that the quantity of grit could also be reduced and with this purpose in a length of half a furlong, the quantity of precoated grit was reduced from 4.4 cu. ft. per 100 sq. ft. to 4.0 cu. ft. per 100 sq. ft. This surface was also found satisfactory and provided a uniform riding surface.

3.2.4. Precoated grit for second coat painting

3.2.4.1. Test lengths were laid in miles 23 and 30 using bitumen precoated chips (¼ in. size) and Mexphalt 80/100 in second coat surface dressing over a surface where first coat had been done with Road Tar, Grade R.T-2 and ½ in. stone chips about a year ago. In both the cases, control lengths with normal chips were also laid side by side for comparison. The quantity of binder was kept at 20 lb per 100 sq. ft. where precoated chips were used and 22 lb per 100 sq. ft. for normal work.

The surfaces were kept under observation and it was seen that the fly-off in case of precoated chips was much less, Photo 3. In mile 30, the grit thrown off by traffic was collected from the berms and measured which showed that the fly-off in precoated chips was only 5 per cent whereas in case of uncoated chips it was 20 per cent. In both the cases, the chips blown off by traffic were not brushed back to the painted surface for the purpose of noting down the fly-off. It is, however, felt that the net loss of grit by whipping off may be between 10-15 per cent in the latter case, if normal operation of regularly putting the blown off grit back on surface is adopted. These surfaces are still under observation for their life though it is more or less definite that the life of portion treated with precoated chips will be more due to greater quantity of gritting material adhering to the surface.

3.2.5. There is no difficulty in spreading precoated chips by spinning basket method as both tar coated and bitumen coated chips are non-tacky at normal application temperature. Both tar and bitumen can thus suitably be used for precoating depending upon the availability of binder.

The cost of binder used in precoating can be adjusted by reducing the quantity of binder in the surface dressing operation and hence the only additional cost involved is that of the labour for precoating which is about Rs 20 per 100 cu. ft. of the precoated grit Appendix II. This cost in some cases can be met by reducing the quantity of chipping material and even if this is not possible, it is worthwhile spending some extra money for this operation keeping in view the net saving by delayed renewals.

3.2.6. Precoated brick grit

3.2.6.1. Precoating of brick grit with both road tar and bitumen tried in laboratory in the first instance showed that the quantity of binder required was as high as 3 per cent for uniform coating of brick chips due to its absorptive nature. Since this quantity of binder would make the operation very expensive, necessity was felt for thinning the binder before mixing in order to give a uniform but thin coating with a lower quantity of binder.

3.2.6.2. Precoating of brick grit was tried using Tar and Bitumen thinned with an equal quantity of waste mobile oil and it was seen that the chips were coated fairly uniformly even with 1 per cent tar or bitumen.

3.2.6.3. In order to see the effect on the physical properties, the brick grit was tested for aggregate crushing value and aggregates

Photo 1

Showing preparation of precoated chips by hand-drum mixer

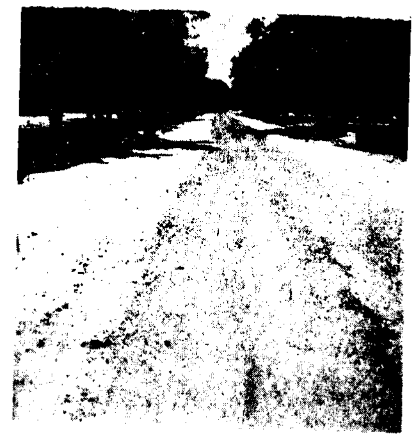

Photo 2

Showing first coat painted surface in mile 42; the portion in the foreground is with the normal grit and in the back-ground with precoated grit

Photo 3

Showing second coat painted surface in mile 23; the portion in the foreground is with normal grit and in the back-ground with bitumen coated chips

impact value using different quantities of binders for precoating. The results of these tests are given in Table 1. The samples after these tests were examined and it was seen that there was no adhesion of the particles due to presence of binder and the test results could be taken to be representative of the actual quality of grit.

TABLE 1

Showing properties of precoated brick grit

Sl No.	Precoating treatment	Agg. crushing value (per cent fines)	Agg. impact-test (per cent fines)
1.	Brick grit uncoated	33.1	27.3
2.	Brick grit coated with 1 per cent tar and 1 per cent mobile oil	26.7	18.4
3.	Brick grit coated with 2 per cent tar and 2 per cent mobile oil	25.2	17.5
4.	Brick grit coated with 3 per cent tar and 3 per cent mobile oil	21.8	13.8
5.	Brick grit coated with 3 per cent tar alone	23.3	9.5
6.	Brick grit coated with 1 per cent bitumen and 1 per cent mobile oil	27.2	11.3
7.	Brick grit coated with 2 per cent bitumen and 2 per cent mobile oil	24.0	9.4
8.	Brick grit coated with 3 per cent bitumen and 3 per cent mobile oil	25.3	7.3
9.	Brick grit coated with 3 per cent bitumen and 1 per cent mobile oil	23.4	9.3
10.	Brick grit coated with 3 per cent bitumen alone	22.4	7.0

From the above results, it is seen that the quality of brick grit is considerably improved by precoating and it may be possible to use brick grit coated with 1 per cent tar or bitumen thinned with mobile oil in place of stone chips for surface dressing. The cost of such a grit in many places in U.P. will be lower than that of stone chips and will thus effect economy in road construction, Appendix III. The quantity of binder in these cases may be kept as 40 lb per 100 sq. ft. with ½ in. chips in first coat painting and 20 lb/100 sq. ft. with ¼ in. chips in second coat painting.

3.2.6.4. Test lengths were laid in mile 33 using bitumen precoated chips ($\frac{1}{4}$ in. size) and Mexphalt (80/100) in second coat painting over the surface where first coat painting had been done with tar R.T.-2 and $\frac{1}{2}$ in. stone chips over stone water-bound. The brick grit was precoated with 1 per cent Mexphalt (80/100) mixed with an equal quantity of waste mobile oil. The quantity of grit and binder was same as mentioned in para 3.2.4.1. The surface is behaving satisfactorily so far and no undue crushing has been observed.

3.3. Built up spray-grout specification with brick metal

3.3.1. Built-up spray grout specifications as tried in the Punjab[1] are given in Appendix IV. These specifications were laid side by side with the normal two coat painting over W.B.M. using brick metal in mile 28 of Lucknow-Sultanpur road during August, 1958. Road Tar Grade R.T.2 was used in first coat at 22 lb per 100 sq. ft. covered by $3\frac{1}{2}$ cu. ft. of ($\frac{1}{2}$ in. - $\frac{3}{4}$ in.) granite stone chips per 100 sq. ft. spread in two grades as required. The sealing coats (viz., second and third painting coats) were given using mexphalt 80/100 18 and 20 lb/100 sq. ft. respectively and stone chips at $2\frac{1}{4}$ cu. ft. (1/4 in. – 3/8 in.) and $1\frac{1}{4}$ cu. ft. (1/8 in. – 1/4 in.) per 100 sq. ft. respectively.

3.3.2. For the normal painting work on brick metal W.B.M. the consolidation of brick metal was done in September, 1958 followed by first coat painting with 45 lb/100 sq. ft. of Road Tar Grade R.T.2 and 4.4 cu ft. of $\frac{1}{2}$ in. stone chips in October, 1958. The second coat was given in May, 1959, using Mexphalt 80/100 at 22 lb/per 100 sq. ft. and 2.2 cu. ft. of 1/4 in. stone grit.

3.3.3. The surface obtained by built-up spray grout method was far superior to the surface obtained in normal work both after first coat painting and second coat painting. No patches were formed on the former surface whereas patching had to be done at places in normal work after first coat painting.

3.3.4. It would be seen that the quantity of binder in built-up spray grout work is 60 lb and that of grit 7 cu. ft. per 100 sq. ft. In normal, two-coat work, the quantity of binder and grit is 66 lb and 6.6 cu. ft. respectively. There is an overall saving of 4 per cent by using built-up spray grout specification and the surface obtained is far better than obtained in normal work. Details of cost are given in Appendix III. The loss of grit is very much reduced due to immediate covering of first and second coat grit.

3.4. Simultaneous coat of tar and bitumen for surface painting

3.4.1. As discussed in para 1.4, it was thought that if tar and bitumen could be applied simultaneously one after another in a

single operation (grit being used only once), it might be possible to derive advantage of both tar and bitumen and thus delay the necessity of application of the second coat which in many cases may be difficult to give in a year or so.

3.4.2. Experiments were conducted in the laboratory to see the miscibility of tar and bitumen and it was noted that both the products were miscible in any proportion and also there was no separation between the layer of tar and bitumen when two blocks, one having bitumen applied on its face and the other having tar on its face, were joined together and pulled out.

Experimental lengths were laid on two roads in 1953 using tar-mexphalt and liquid asphalt-mexphalt simultaneously in separate stretches[6]. The results of behaviour of these surfaces indicated that some of these portions stood well for about four years and riding quality of these portions was better than two-coat work. A renewal coat was given after four years.

3.4.3. In order to confirm these findings and to see whether the quantity of binder used can be further reduced to make the treatment cheaper, an experimental stretch was laid in miles 32-33 of Lucknow-Sultanpur road.

3.4.4. The brief specifications were as follows :

Road Tar Grade R.T. 2 at 35 lb per 100 sq. ft. was spread over the W.B.M. surface and allowed to remain on the surface for a period of 4 hours, during which time patries were kept watered to keep down dust. After this period, mexphalt 80/100 was spread at 25 lb/100 sq. ft. followed by granite chips at 4.5 cu. ft. per 100 sq. ft. and rolled. It was noted that this quantity of binder was rather excessive and in a further stretch the quantity of tar and mexphalt was reduced to 30 lb and 20 lb per 100 sq. ft. respectively.

3.4.5. These surfaces have been under observation for over two years. In the portion where larger quantity of binder was used, bleeding took place and sand blinding had to be done. The behaviour was otherwise satisfactory and much better than that of adjoining stretches where first coat painting with tar at 44 lb and grit at 4.4 cu. ft. 100 sq. ft. had been given. The experiment is still under observation to assess the life of this treatment, but results indicate that such specifications can be usefully adopted.

3.5. Use of diluted emulsion in W.B.M. road construction

3.5.1. Diluted emulsion has been used in U.S.A. as a dust palliative on earth roads, the emulsion being added in water and applied to the road surface in several applications[4]. The results were reported to be effective and it was noted that a stabilized base was built-up capable of taking fairly good amount of traffic.

3.5.2. The necessity of frequent blinding of W.B.M. surface has already been discussed in para 1.5. It was, however, thought that if diluted emulsion could be used during consolidation work, it might be possible to improve the stability of the top crust and reduce the dust nuisance and at the same time obviate the necessity of frequent blinding.

3.5.3. As is the usual practice in water-bound road construction, local soil is used as a binding material while consolidating the road metal. One of the methods of introducing binding material is to prepare a slurry of local soil and water and after dry rolling, pour it on the surface followed by wet rolling. Sufficient quantity of slurry to fill up all the voids is used. The surface is finally blinded with earth next day and then opened to traffic. If, therefore, emulsion is mixed with this slurry before it is poured on the surface, it is possible to stabilise this slurry in the voids and on top, thereby making the crust more stable and resistant to movement by traffic.

3.5.4. Short experimental lengths were laid in mile 35 as per details below :

After dry rolling of stone, slurry was mixed with bitumen emulsion (H.R.M. emulsion containing about 60 per cent bitumen). It was diluted by water in the ratio 1 : 19 and then the local earth was put in the diluted emulsion in drums and mixed and agitated to have uniform slurry. The quantities of earth and emulsion were controlled so as to limit the quantity of emulsion to 0.1 gallon per sq. yd. of the treated surface. (Approximately 5 cu. ft. of earth was required per 100 sq. ft. of W.B.M. surface and the quantity of emulsion was 1.1 gallon or 11 lb per 100 sq. ft.)

After pouring this emulsion slurry, the surface was rolled as usual. Next day the surface was blinded with local earth and opened to traffic.

In another length, ordinary earth slurry was poured in so as to fill up the voids upto $\frac{1}{2}$ in. to $\frac{3}{4}$ in. from the top of the surface. Then the emulsion slurry containing emulsion at the rate of 0.1 gallon per sq. yd. of surface was poured on top so as to fill up the voids completely and then the surface was rolled as usual. In this way, the concentration of bitumen was increased in the top 1 in. layer of W.B.M. surface. The surface was opened to traffic next day after blinding with earth.

In other lengths, use was made of fine local sand and coarse sand, as a filler in place of local earth. In this case, the binding material (sand) was spread on W.B.M. surface at the rate of 5 cu. ft. per 100 sq. ft. and then diluted emulsion was spread on top limiting the application of emulsion to 0.1 gallon per sq. yd. followed by rolling so that sand worked inside the voids. The surface was opened to traffic, after final rolling.

3.5.5. These surfaces were observed regularly under traffic. It was noted that the surface with fine sand began to break up indicating that fine sand had not been stabilized. The surface with coarse sand, however, behaved satisfactorily for about two months but then it also began to break up under traffic.

The portions where local earth was used with emulsion behaved satisfactorily and no patches developed. No blinding was resorted to for a period of six months. As painting over this surface could not be done at this stage, a sprinkling of diluted emulsion at the rate of .01 gallon per sq. yd. followed by blinding with local earth was resorted to. This treatment kept the surface intact for a further period of four months when it was covered by surface dressing.

3.5.6. The experiment thus shows that diluted emulsion can be used for stabilizing the W.B.M. surface. The details of the cost are given in Appendix III. It also indicates the possibilities of treating the W.B.M. surface with emulsion where surface dressing is not to be done and the surface is to be maintained as such. It might also cut down the dust nuisance to some extent on W.B.M. surface.

4. CONCLUSIONS

4.1. The use of brick grit for surface painting offers a good substitute for stone grit and may result in considerable economy where stone grit is costly to obtain.

4.2. By precoating the brick grit, considerable improvement in its aggregate impact value is achieved. Aggregate crushing value is also improved. Thus its use after precoating can be made for surface dressing work.

4.3. Precoating of ordinary stone grit by tar or bitumen improves the adhesion of grit with binder on surface and thus loss of grit is minimised.

4.4. Use of built-up spray grout specification using brick metal results in much better surface and minimises to great extent the possibilities of crushing of brick metal. The loss of grit is considerably reduced.

4.5. Simultaneous coat of tar and bitumen for surface dressing gives better surface and can be tried where one coat of surface dressing is to be given for lack of funds.

4.6. Use of diluted bitumen emulsion in W.B.M. surface obviates the necessity of frequent blinding and helps in upkeep of surface and results in a better surface.

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Appendix I

Location of test lengths on Lucknow-Sultanpur Road

Sl. No.	Brief specifications	Location (mile)	Length of trial	Period of laying test length
1	Use of brick grit for surface painting			
	(a) First coat painting	22 29	½ furlong 1 furlong	November 1957 November 1958
	(b) Second coat painting	22 12	½ furlong ½ furlong	November 1958 June 1959
2	Use of precoated grit for surface dressing			
	(a) First coat painting	42	2 furlongs	March 1959
	(b) Second coat painting	23 30	2 furlongs 4 furlongs	March 1959 July 1959
	(c) Precoated brick grit in second coat painting	33	1 furlong	May 1960
3	Built-up-spray grout specifications for brick metal	29	1 furlong	August 1958
4	Simultaneous coat of tar and bitumen for surface painting	32 33	3 furlongs 1 furlong	May 1958
5	Use of diluted emulsion for W.B.M. road construction	35	1½ furlongs	September 1958

Appendix II

Traffic intensity on Lucknow-Sultanpur Road
(Seven days census taken during 1958)

Mile No.	Animal driven (tonnage)		Power driven (tonnage)		Total intensity (tonnage)	
	Maximum	Average	Maximum	Average	Maximum	Average
4	330	258	1,028	876	1,358	1,134
14	303	234	909	736	1,212	970
21	95	77	801	687	896	764
35	255	183	721	585	976	768
44	154	131	560	494	714	625

Appendix III

ANALYSIS OF RATES

1. COST OF BRICK GRIT. (with an average lead of 3 miles)
Task 130 cu.ft.

A. I coat grit—$\frac{1}{2}$ in. size ($\frac{1}{4}$ in.—$\frac{3}{4}$ in).

		Rs
Cost of overburnt bricks at kiln site—1,200 Nos. at Rs 25 per 1,000 Nos.	=	30.00
Cartage from brick kiln to site of work (distance 3 miles) (Rate of cartage as per schedule) 1,200 Nos. at Rs 8.30 per 1,000 Nos.	=	9.96
Breaking charges (10 beldars at Rs 1.50 each)	=	15.00
Stacking at site	=	1.75
Total rate	=	56.71

Say Rs 56.70 per 100 cu. ft.

B. II Coat grit $\frac{1}{4}$ in. size (1/8 in.—3/8 in.)

		Rs
Cost of overburnt brick at kiln site—1,200 Nos. at Rs 25 per 1,000 Nos.	=	30.00
Cartage from brick kiln to site of work (distance 3 miles) (Rate of cartage as per schedule) 1,200 Nos. at Rs 8.30 per 1,000 Nos.	=	9.96
Breaking charges (beldars 15 Nos. at Rs 1.50 each)	=	22.50
Stacking at site	=	1.75
Total rate	=	64.21

Say Rs 64.20 per 100 cu. ft.

COST OF PRECOATED STONE GRIT

A. $\frac{1}{2}$ in. size ($\frac{1}{4}$ in.—$\frac{3}{4}$ in.) Gauge for first coat work task 100 cu. ft.

		Rs
Cost of ordinary grit 100 cu. ft. at Rs 125 per cu. ft.	=	125.00
Cost of tar at 0.8%, i.e. $\frac{100 \times 90 \times .8}{100}$ = 72 lb at Rs 375 per ton	=	12.00
Labour charges for precoating the grit in hand drum mixers including screening (as per actual work at site) at Rs 20 per 100 cu. ft.	=	20.00
Hence total cost per 100 cu. ft.	=	157.00

		Rs
B. ¼ in. size-(1/8 in.—3/8 in.) Gauge—for 2nd coat work. Cost of ordinary grit 100 cu. ft. at Rs 130 per 100 cu. ft.	=	130.00
Cost of Mexphalt 80/100 at 0.8 per cent, i.e., 72 lb at Rs 425 per ton	=	13.50
Labour charges for precoating the grit	=	20.00
Total rate	=	163.50

Hence rate for 100 cu. ft. = Rs 163.50

3. COST OF PRECOATED BRICK GRIT

		Rs
A. ½ in. size for first coat work with tar and old mobile oil (Task 100 cu.ft.)		
Cost of Brick grit 100 cu.ft. at Rs 56.70 per 100 cu. ft. (as per item 1-A)	=	56.70
Cost of tar at 1 per cent, i.e., $\frac{100 \times 85}{100}$ = 85 lbs. at Rs 375 per ton	=	14.23
Cost of old mobile oil at 1 per cent i.e. 85 lb or 8.5 gallons at Rs 0.50 per gallon	=	4.25
Labour charges for precoating the grit	=	20.00
		95.18

Say Rs 95.00 per 100 cu.ft.

		Rs
B. ¼ in. size for 2nd coat work with bitumen and old mobile oil. Task—100 cu. ft.		
Cost of brick grit 100 cu. ft. at Rs 64.20 per 100 cu.ft. (as per item No. 1-B)	=	64.20
Cost of binder (Mexphalt 80/100 at 1 per cent = $\frac{100 \times 85}{100}$ 85 lb at Rs 425 per ton	=	16.12
Cost of old mobile oil (as in A above)	=	4.25
Labour charges for precoating the grit	=	20.00
Total rate		104.57

Say Rs. 104.50 per 100 cu. ft.

4. BUILT UP SPRAY GROUT SPECIFICATION
Task 100 sq. ft.

(I) **Materials**

		Rs
(1) Brick ballast 1 × 100 × 3/8 = 37½ cu. ft. at Rs 45 per 100 cu. ft.	=	16.87
(2) Grit ½ in. size at 3.5 cu.ft. = 3.5 cu.ft. at Rs 125 per 100 cu.ft.	=	4.37
(3) Grit ¼ in. size—(1/8 in.-3/8 in.) 3.5 cu.ft. at Rs 130 per 100 cu.ft.	=	4.55
(4) Binder (a) Tar RT-2 at 22 lb = 22 lb at Rs 375 per ton	=	3.70
(b) Mexphalt 80/100 at 38 lb = 38 lb at Rs 425 per ton.	=	7.20
		36.69

B. **Labour charges**

		Rs
(1) Consolidation including first coat paint at Rs 3.25 per 100 sq. ft.	=	3.25
(2) Second and third coat painting at Rs 0.75 per 100 sq. ft.	=	0.75
Total		40.69

Say Rs 40.70 per 100 sq. ft.

(II) **Two coat work over brick ballast**

A. **Materials**

		Rs.
(1) Brick ballast 1 × 100 × 3/8 = 37½ cu.ft. at Rs 45 per 100 cu. ft.	=	16.87
(2) Grit ½ in. size at 4.4 cu.ft. at Rs 125 per 100 cu. ft.	=	5.50
(3) Grit ¼ in. size at 2.2 cu.ft. at Rs 130 ,, ,,	=	2.86
(4) (a) Binder tar at 45 lb at Rs 375 per ton	=	7.53
(b) Mexphalt at 22 lb at Rs 425 per ton	=	4.18
		36.94

B. **Labour**

		Rs
(1) Consolidation B. Ballast 37½ cu. ft. at 7.50 per 100 cu. ft.	=	2.81
(2) (a) Surface painting first coat at Rs 1.75	=	1.75
(b) ,, ,, second coat at Rs 0.87	=	0.87
		42.37

Say Rs 42.40 per 100 sq. ft.

per cent saving of (I) over (II) = $\frac{42.40-40.70}{42.40}$
= 4 per cent

5. EXTRA COST FOR USE OF DILUTED EMULSION IN WATER BOUND CONSTRUCTION
Task 100 sq. ft.

		Rs
Cost of emulsion at 0.1 gallon per sq. yd. or 1.1 gallon or 11 lb per 100 sq. ft. at Rs 400 per ton	=	1.96
Extra labour charges for preparation of diluted emulsion and slurry etc. (as per actual work at site)	=	0.15
Total		2.11

Say Rs 2.10 per sq.ft.

Appendix IV

Built-up spray grout specification for the consolidation of brick metal

Over the existing soling coat and between mud walls (medhis) a layer of one inch good clay is spread. Over this 90 per cent of the brick metal (retaining the 10 per cent neat metal from top) is spread by spinning basket method ensuring uniform spreading. The brick metal is rolled dry starting from the sides, removing unevenness of the surface by adding 10 per cent reserve of the neat metal kept for the purpose. As soon as the surface is correct to template and the brick metal is fairly interlocked (avoid crushing of metal by excessive rolling) water is sprayed so that the top surface becomes fairly moist. The surface is allowed to air and as soon as the surface is skin dry, Road Tar RT-2 is spread at the rate of 22 lb per 100 sq. ft. and covered with $3\frac{1}{2}$ cu.ft. of $\frac{1}{4}$ in. —$\frac{3}{4}$ in. grit per 100 sq. ft. spreading in two grades—$\frac{3}{4}$ in. to $\frac{1}{2}$ in. (70 per cent) rolled and followed by $\frac{1}{2}$ in. to $\frac{1}{4}$ in. gauge (30 per cent). It is rolled lightly till the grit is well pressed.

Water is then added and consolidation is done as water-bound. Rolling should be stopped when the clay from the bottom has risen up to just about $\frac{3}{4}$ in. from the surface, Next day, the inequalities of the surface are made up with premixed grit and the surface sealed with two coats of binder (using either tar or bitumen) and grit using following quantities.

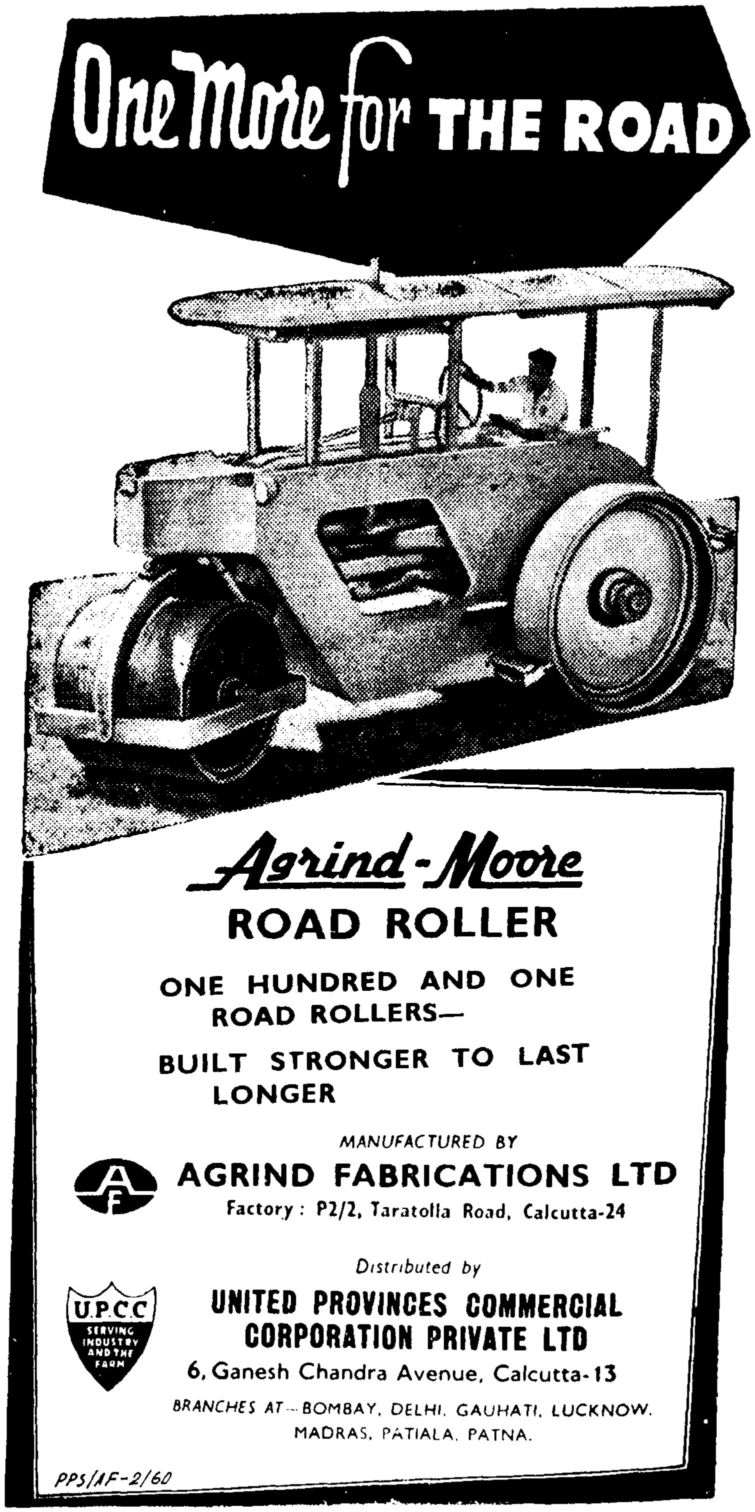
	Binder Lb per 100 sq. ft.	Grit Cu. ft. per 100 sq. ft.
II coat	18	$2\frac{1}{4}$ ($\frac{1}{4}$ in. to $\frac{3}{8}$ in. gauge)
III coat	20	$1\frac{1}{4}$-$1\frac{1}{2}$ ($\frac{1}{8}$ in. to $\frac{1}{4}$ in. gauge)

The third coat is given the same day as the second coat.

No traffic should be allowed on the surface for about a week during which time the berms, etc., should be completed. While allowing the traffic, only pneumatic tyred traffic should be allowed for two to three days. The segregation of traffic should be done so as to cause the traffic to traverse the entire width of road uniformly.

REFERENCES

1. Chaturvedi, D.C. Bituminous Treatment of Brick Metal Water Bound Macadam Surface in U.P.—Journal of Indian Roads Congress Vol. XIX-1 & 4, 1955.
2. Chaturvedi, D.C. Use of Overburnt Brick Metal for Low Cost Road-construction—ECAFE Seminar on low cost roads and soil stabilization, January 1958.
3. Dew Hurst, J.R. and Cantlay, L.G. Surface dressing with tar coated chippings—Road Tar, December 1955.
4. Wood, T.K. Texas Experience with Diluted Emulsions for Flexible base, World Construction—June 1957.
5. Chaturvedi, D.C. & Tayal J.C. Brick Pavement in Highway Construction, Journal of Indian Roads Congress—Vol. XXIII-3, December 1958.
6. Annual Report of U.P., P.W.D., Research Institute, 1953-54.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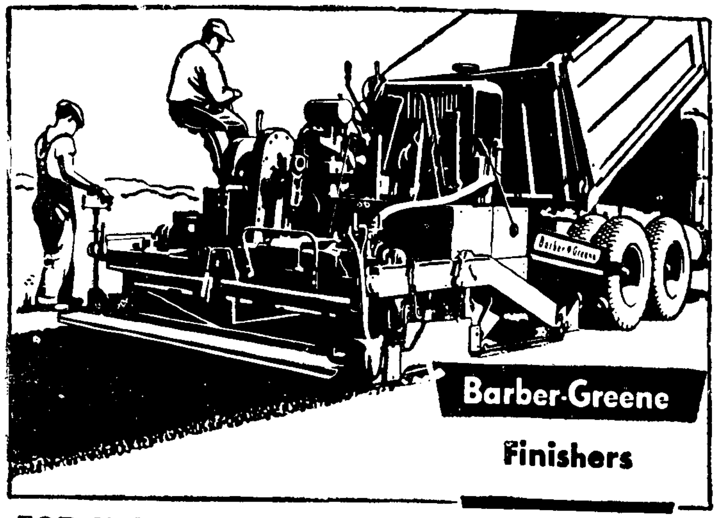

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