1055

57

The Indian Roads congress

Bulletin No. 4.

1957

THE INDIAN ROADS CONGRESS

ROAD RESEARCH

Bulletin No. 4

Issued by

THE INDIAN ROADS CONGRESS,
Jamnagar House, Shahjahan Road, New Delhi-2.

Price: Rs 5 or 7 Sh.

The Indian Roads Congress as a body does not hold itself responsible for statements made, or for opinions expressed, in this Bulletin.

CONTENTS

" A NEW METHOD OF DETERMINING LIQUID LIMIT OF SOILS "

By

H. L. UPPAL AND H. R. AGGARWAL*

SUMMARY

A cone penetrometer has been evolved for determining the liquid limit of soils with a fairly wide range. The cone has a half angle of 15 degrees 30 minutes and penetrates to a depth of 1 inch under a load of 148 grams when the soils are at liquid limit.

INTRODUCTION

Liquid limit is one of the important tests in Soil Mechanics. It gives the moisture content at which the soil passes from the plastic to the liquid state and has therefore lost practically all its strength. The liquid limit of soils generally increases with increase in clay content.

The test for determining liquid limit was initially developed by Albert[1] Atterberg, a Sweedish Agronomist, in 1911 and used for classifying soils for agricultural purposes. With the advancement of soil science, the use of soil as a material of construction began to be recognised and in 1926, Wintermyer[2] suggested the adoption of liquid limit as one of the soil constants to evaluate its properties as an engineering material. Since then, this soil constant is being widely used by highway engineers to evaluate soils for use as subgrade, embankment, base-course material, etc.

METHODS OF DETERMINING LIQUID LIMIT

1. **Hand Method:** The determination of liquid limit as originally proposed was carried out by what is commonly known as 'Hand Method.' This consists in taking about 30 grams of soil in a china clay-dish, mixing some water and levelling the thick paste with the spatula. A standard groove (5/64 in. in width) is cut in the middle of the paste and the dish is jarred against the palm of the other hand till the groove closes down. The moisture percentage when the groove closes at 10 strokes is taken as the

---

* Central Road Research Institute, New Delhi.

liquid limit. This method is one of the recommended methods for liquid limit (A.S.T.M. Designation D 423-39). On account of ease of handling, it has been widely adopted, especially in routine testing when large number of soil samples are required to be tested.

2. **Mechanical Device**: With further development in the field of soil mechanics, the need for a more precise test procedure became a necessity and a mechanical device was developed by Arthur Casagrande[3] in 1932 which could give good reproducibility of results.

The mechanical method requires at least three random trials at different moisture contents and the corresponding number of blows required to close the standard groove is recorded. The percentage of moisture and the number of blows are plotted on a semi-log scale and a flow curve is drawn. The moisture content corresponding to the point where the flow curve intersects the 25 blow line is taken as the liquid limit. The method has been accepted as standard by A. A. S. H. O.[4], B. S. S.[5] and A. S. T. M. From 1954 onwards, this method has even been published as tentative by the A.S.T.M.

On account of personal errors involved, the hand method has now come to stay only as an alternative method as per latest A.S.T.M. standards[6], 1954 and has to be calibrated against the mechanical device.

Though the mechanical device is a marked improvement on the hand method, yet it has its own drawbacks. While testing large number of alluvial soils with this device, it has been observed that liquid limit cannot be determined correctly when the soils are sandy (liquid limit below 25 or so and plasticity index between 4-7). This difficulty was also experienced by Casagrande himself and he attributed it to "insufficient[3] adhesion of such soils to the cup and also to their excessive permeability which causes excessive water content in the region of the groove."

The range of soils for which the mechanical device does not give correct values of liquid limit was determined by preparing soils of decreasing plasticity index, by adding increasing percentage of graded sand to a soil of known plasticity index and then determining their liquid limit. (The plastic limit was determined by the standard method). The analysis of the graded sand along with the grain size distribution is given in Fig. 1. It has been assumed that when graded sand is added to a certain soil, the plasticity index falls proportionately to the per cent of sand added. While dealing with similar subject it has been shown by

Sieve No. (B.S.S.)	Percentage passing
36	100.0
52	81.0
72	60.0
100	43.0
150	20.0
200	1.0

GRAIN SIZE DISTRIBUTION CURVE

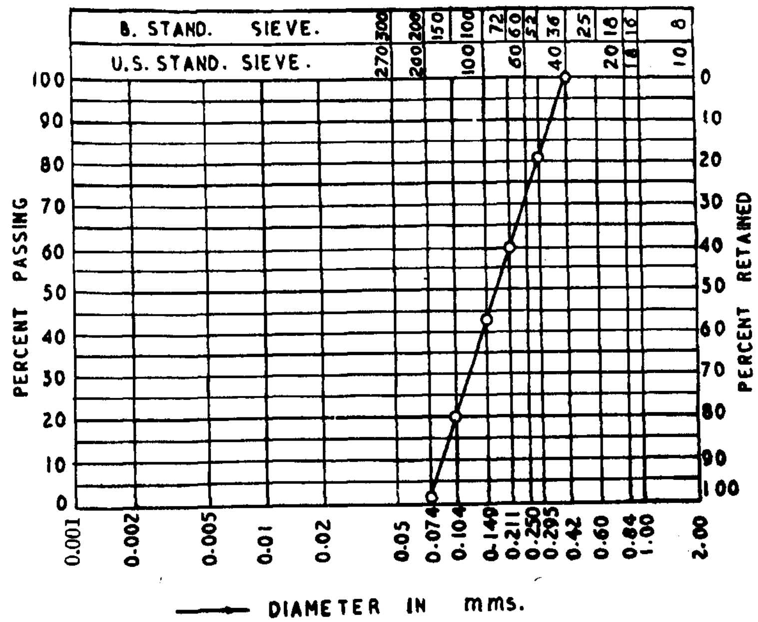


Fig. 1. **Particle size distribution of the graded sand used in blending of soils**

Hogentogler* that plasticity index falls proportionately to the addition of sand, though the accuracy of the results becomes less as the percentage of sand added increases above 45. The values of plasticity index thus calculated according to the above assumption and the one obtained after determining liquid limit and plastic limit are given in Table 1 and also shown in Fig. 2.

* Hogentogler's formula (Ref: Engineering properties of soil by C. A. Hogentogler, p. 273 & Public Roads, Vol. 7, No. 12, Feb., 1927).

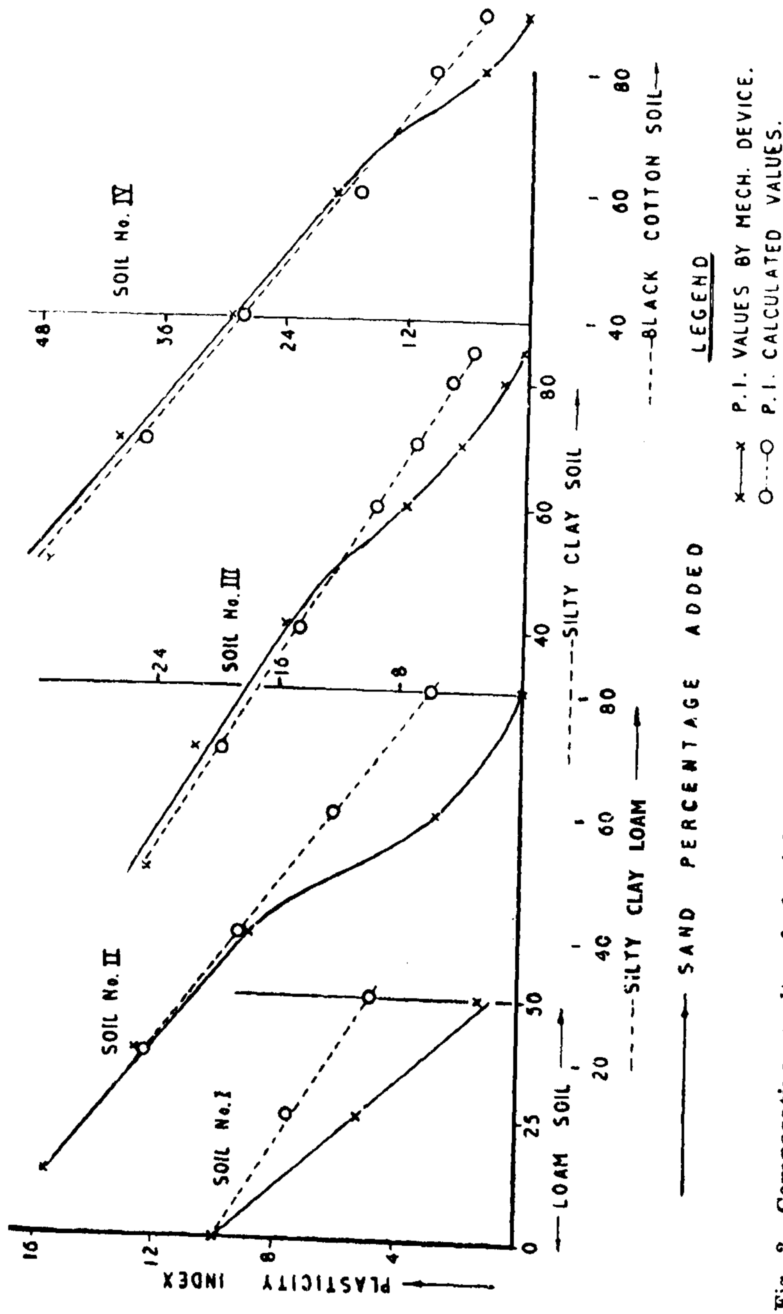


Fig. 2. **Comparative results of plasticity index from liquid limit determined by Casagrande's mechanical device and corresponding calculated values**

TABLE 1

Comparative results of plasticity index obtained from the L.L. determined by Casagrande's Mechanical Device and calculated values on the basis of addition of sand

Soil type	Per cent sand added	Liquid Limit per cent (Mech. Device)	Plastic Limit per cent	Plasticity Index	Plasticity Index by calculation
Loam	Nil	28.0	17.9	10.1	
	25	22.2	16.9	5.3	7.6
	50	17.2	15.9	1.3	5.0
Silty clay loam	0	34.9	19.2	15.7	
	20	29.4	16.9	12.5	12.6
	40	26.0	16.9	9.1	9.4
	60	20.4	17.7	2.7	6.3
	80	18.7	18.7	N.P	3.1
Silty clay	0	44.6	19.9	24.7	—
	20	38.1	16.3	21.8	19.8
	40	31.5	15.7	15.8	14.8
	60	23.7	15.8	7.9	9.9
	70	20.5	16.3	4.2	7.4
	80	19.1	17.4	1.7	4.9
	85	18.1	17.9	0.2	3.7
Clay (black cotton soil)	0	79.0	31.9	47.1	
	20	66.0	25.7	40.3	37.7
	40	50.6	21.4	29.2	28.3
	60	36.0	16.9	19.1	16.8
	80	23.6	18.7	4.9	9.4
	90	19.5	18.8	0.7	4.7

It appears from the results in Table 1 that for sandy soils with plasticity index below 8 to 9, the liquid limit values when determined by Casagrande's device are low and consequently give lower values of plasticity index of the soil.

Since there are large deposits of alluvial soils in India with plasticity index generally ranging from 0–12 (liquid limit below 25 or so) the mechanical device cannot be confidently used for determination of their liquid limit. As has already been pointed out, the hand method involves lot of personal errror and therefore cannot be free from criticism. It, therefore, requires another type of mechanical device which can give reproducible results and yet be applicable to wider range of soils, more so on the sandy side for which existing mechanical device is not found very suitable.

In the mechanical device as well as in the hand method it has been assumed that the force resisting the deformation of the sides of the groove is the shearing[3] resistance of the soil and hence the number of blows required to close the groove of a soil paste represents a relative measure of the shearing resistance of the soil at that moisture content. The soils at their liquid limit moisture content, though said to have lost practically[7] all their strength, yet possess a certain small shearing strength. This strength is a definite value and is same for all soils at their respective[3] liquid[8] limits. Skempton[9] and Northey have also estimated that at liquid limit moistures, clayey soils have a shearing yield stress of the order of 0.12–0.22 lb per sq. inch. Therefore, with the object of evolving a suitable mechanical device, it was kept in view to select some suitable shear test device for determining the liquid limit of soils. The problem was originally started in 1953 but could not be actively pursued on account of pressure of more urgent work.

INITIAL TRIALS

Out of the many methods used for the determination of shear strength, the penetration method is the simplest and most convenient one to use. A trial was made to determine the resistance offered by soils to the penetration of a metallic cylindrical rod 5 cms. long and 1 cm. in diameter. It was fitted to an ebonite rod with a disc for carrying loads. The soil for the test was taken in a cylindrical mould, with 2 in. diameter and 2 in. in depth, the size of which was fixed after a number of trials to ensure that the resistance to penetration was not affected by the sides of the container. It was found that the total weight of the plunger varied between 200 to 300 gms. for a penetration of one inch to get liquid limit moisture content. The total resistance offered by the soils depends not only on the cross-section area of the face of the plunger but also on the side friction which varies from soil to soil and also with the depth of penetration. The device was therefore considered not very simple and for subsequent tests, cone penetrometer was used. It has the advantage that area of contact can be readily calculated from the penetration, apart from the fact that side friction is also very small. One of the standard cones which has been succesfully used in the field with the North Dakota has an angle of 7°–45′. A number of experiments were carried out with this cone with increasing load and soil moistures

to determine the load required for a particular penetration at liquid limit moisture content. It was observed that penetration of one inch was brought about by a small load of 50 grams within which the weight of penetrometer itself could hardly be adjusted. Besides, it was noted that even a small change in load caused marked difference in penetration. This cone was, therefore, considered to be rather too sensitive for such tests—and further experiments were conducted with a cone of half angle equivalent to 15°–30′. The details of the various investigations carried out with this cone are discussed in the following pages.

A: Relationship between time and penetration

Different soils ranging from sandy to clayey in nature were taken at about their liquid limit moisture contents and the penetration of the cone was determined under a particular weight at intervals of 10, 20, 30, 40 and 60 seconds. The results obtained are given in Table 2 and further represented in Fig. 3.

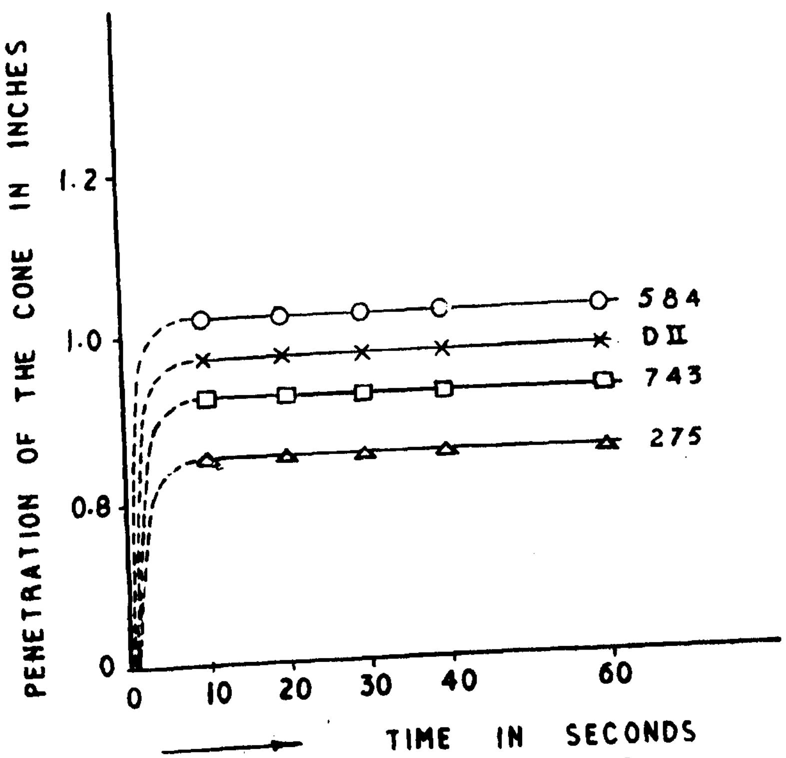


Fig. 3. Effect of time on the penetration of cone

TABLE 2

Effect of time on the penetration of cone

Soil No.	Penetration in inches after				
	10 secs.	20 secs.	30 secs.	40 secs.	60 secs.
D II	0.97	0.97	0.97	0.97	0.97
584	1.02	1.02	1.02	1.02	1.02
275	.85	.85	.85	.85	.85
743	.92	.92	.92	.92	.92

It was observed that maximum penetration takes place within the first few seconds and the results clearly show that no change in penetration takes place after 10 seconds of the release of the cone. However, 30 seconds were taken as a very safe limit and adhered to in all further tests.

B : Relationship between load and penetration at liquid limit moisture contents

Five different soils were selected with plasticity index above 10 (liquid limit ranging from 29 to 74) for which it is accepted that the results obtained by mechanical device are correct. In each case, sufficient quantity of soil was taken, worked into a paste and its moisture content was adjusted at liquid limit as determined by mechanical device. The sample was taken in the metallic trough and penetration of the cone was recorded at increasing loads. After each penetration reading, the cone was cleaned and the soil in the trough was levelled for the next determination. The results obtained are given in Table 3 and also shown graphically in Fig. 4.

The results clearly show that the two variables bear a linear relationship at liquid limit moisture content. The various points for all these soils fall so close that these can be considered to follow one definite straight line. There is, however, small progressive deviation after 0.86 inch penetration and, therefore, for

TABLE 3

Co-relation between load (grams) and penetration (inches) at liquid limit moisture content

Soil No. 743		Soil No. 710		Soil No. 275		Soil No. 584		Soil No. DII	
Load	Penetration	Load	Penetration	Load	Penetration	Load	Penetration	Load	Penetration
106	0.75	106	0.78	116	0.78	106	0.76	106	0.75
126	0.85	126	0.87	126	0.86	116	0.80	116	0.83
136	0.93	136	0.91	136	0.91	126	0.83	136	0.92
141	0.99	146	0.97	141	0.93	136	0.93	141	0.96
146	1.00	151	1.00	146	0.95	146	1.00	146	1.00
156	1.10	156	1.08	156	1.03	156	1.03	156	1.05

purposes of determination of liquid limit, a penetration of 0.86 inch will be ideal. It will, however, be more convenient to record a penetration of 1 in. For all further tests, one inch also being the

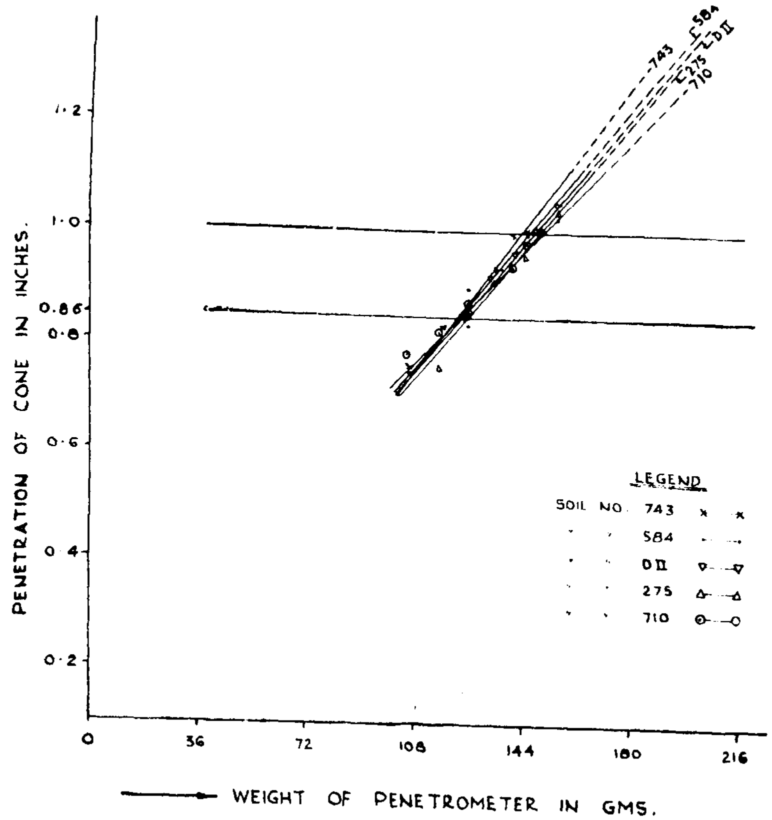


Fig. 4. **Relationship between load and penetration at liquid limit moisture**

simplest and whole number was kept as the standard penetration of cone with half angle of 15°-30'.

The exact corresponding load required to cause 1 inch penetration for different soils was foundfrom Fig. 4 and is recorded

in Table 4 given below :

TABLE 4

Load required for 1 in. penetration of the cone at liquid limit moisture content

S. No.	Soil No.	Soil type	Liquid limit by Casagrande's mechanical device	Load required for 1 in. penetration of the cone
1	743	Black cotton	73.9	146 gms.
2	710	,,	68.0	151 gms.
3	275	Moorum	48.2	151 gms.
4	584	Moorum	37.8	146 gms.
5	DII	Alluvial	29.0	146 gms.

$$\text{Average} = \frac{740}{5} = 148 \text{ gms.}$$

This clearly shows that for 1 inch penetration, the corresponding load required would be 148 grams which in turn will represent liquid limit moisture content.

C: Relationship between load and moisture content for one inch penetration

In this set of experiments, load required for one inch penetration of the cone was determined at varying moisture content on either side of the liquid limit for a number of soils. The results obtained are given in Table 5 and shown graphically in Figure 5.

The results evidently show that the two variables follow a mathematical relationship for a wide range of moisture content. The curve tends to be parabolic with increase in variation on either side of liquid limit moisture content showing that the rate of change in penetration resistance is not constant for all moisture contents. There is only a small range close to the liquid limit at which the variation in load can be considered to follow a linear relation. This range lies between a load of 120 to 160 grams which means corresponding penetration of 0.8—1.1 inches against a load of 148 grams required for penetration of 1 in.

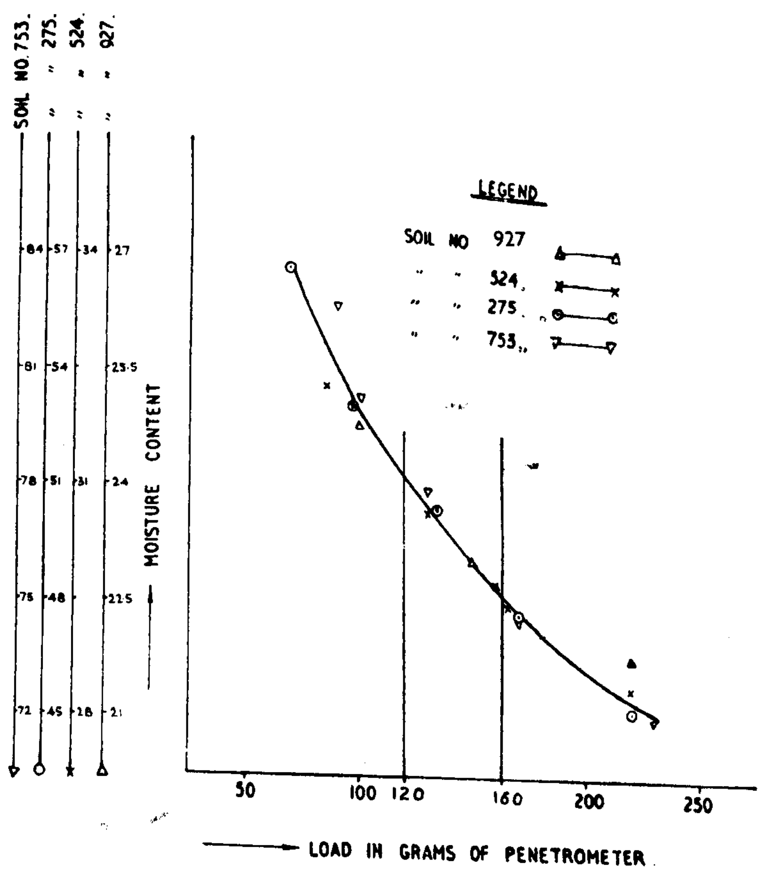
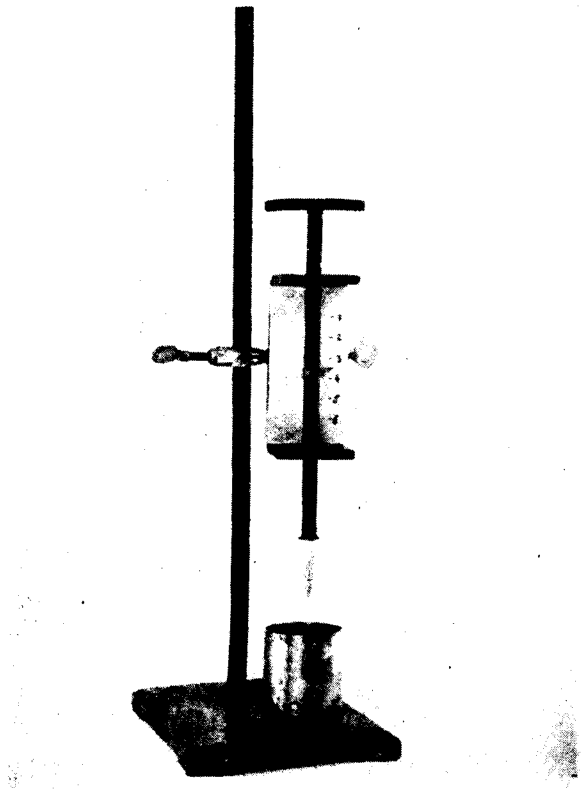


Fig. 5. **Relationship between load and moisture for 1 inch penetration of the cone**

D: Relationship between penetration and moisture at particular load

A large number of soil samples with varying liquid limits were selected and penetration was determined under a constant load of 148 gms. at varying moisture contents on either side of



Penetrometer devised by the Author (Dr. H. L. Uppal) for determining liquid limit of soils

liquid limit so that the penetration of the cone ranges between 0.8 to 1.1 inches only.

S. No.	Soil No.	Soil type	Liquid limit per cent
1	DIV	Alluvial	24.3
2	DIII/20S	Alluvial (sand mix)	29.1
3	524	Moorum	30.3
4	816/20S	Moorum (sand mix)	38.0
5	816	Moorum	45.1
6	275	Moorum	48.6
7	753/20S	Black cotton (sand mix)	64.4
8	753	Black cotton	77.4

The results as given in Table 6 and shown graphically in Fig. 6 clearly show that the two variables bear a straight line relation between the above range.

TABLE 5

Co-relation between load and moisture content for one inch penetration of the cone

Soil No. 927		Soil No. 524		Soil No. 275		Soil No. 753	
Moisture percentage	Load in gms.	Moisture percentage	Load in gms.	Moisture percentage	Load in grams.	Moisture percentage	Load in grams.
21.3	300	28.4	220	45.2	220	72.1	230
21.8	220	29.5	165	47.9	170	74.6	170
22.5	180	29.8	160	50.5	132	78.0	130
23.1	150	30.7	130	53.1	97	80.4	100
24.8	100	32.3	85	56.7	69	82.7	90

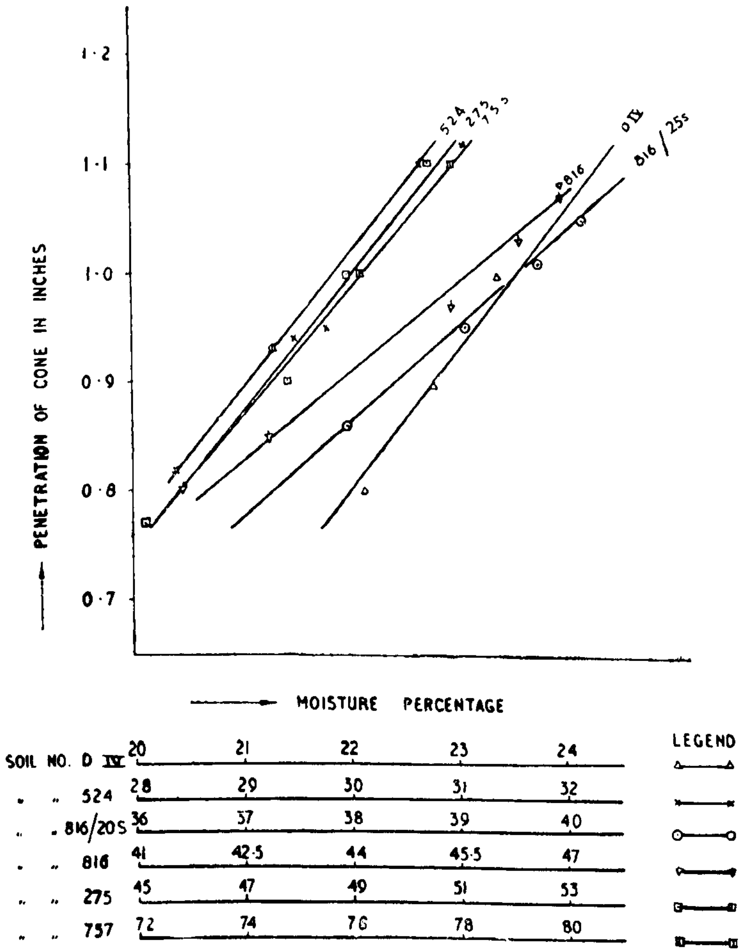


Fig. 6. **Relationship between penetration and moisture content under a particular load**

Having fixed the penetration and the load of the cone device and establishing that all the variables involved in such tests bear a definite relationship close to its liquid limit moisture content, this device was considered suitable to be adopted for determining moisture content at which the soil has a very small but finite shearing strength i.e., its liquid limit stage.

TABLE 6

Soil No.	Moisture Percentage	Mean penetration in inches
D IV	21.3	0.68
	22.1	0.80
	23.4	1.00
	24.0	1.10
524	28.4	0.82
	29.5	0.94
	29.8	0.95
	30.7	1.10
	31.0	1.12
DIII/20S	29.4	0.85
	29.9	0.92
	31.1	0.97
	31.9	1.03
	32.5	1.09
816/20S	36.5	0.76
	38.0	0.86
	39.1	0.95
	39.8	1.01
	40.2	1.05
816	41.7	0.80
	42.9	0.85
	45.4	0.97
	46.5	1.03
	47.1	1.07
275	45.2	0.77
	47.9	0.90
	49.0	1.00
	50.5	1.10
753/20S	60.7	0.75
	61.5	0.82
	64.1	0.94
	65.6	1.00
	68.0	1.08
753	72.1	0.75
	74.6	0.93
	76.3	1.00
	78.0	1.10

DESCRIPTION OF THE APPARATUS

The apparatus which was designed by the senior Author is shown in Photograph, page 15. It consists of an ebonite stem 8 inches in length, the top of which is threaded into an ebonite disc while the lower end is capped with a metallic cone of half angle of 15 degrees 30 minutes, 1.20 inch coned length. The stem passes through a U-shaped steel holder which has two holes, 8 cms. apart, one at the upper and the other at the lower end. The size of the holes is such that it keeps the stem vertically in position besides permitting its free movement. The stem is provided with a sliding pointer which can be adjusted at any point over the scale on the holder. This cone arrangement weighs 148 grams inclusive of the top disc, stem and the pointer attached to it. The apparatus is clamped tightly to an iron stand. The soil paste is prepared in a china clay dish with a spatula and is then transferred to a small cylindrical metallic trough 2 in. inner dia., and 2 in. in height for carrying out the test.

RECOMMENDED PROCEDURE

About 150 grams of the soil sample, passing 40 mesh Indian or U.S. Std. sieve, shall be taken and worked into a paste with the addition of water. (In case of clayey soils, it shall preferably be kept for an hour or so in the humidity chamber, about 95 per cent humidity to ensure uniform distribution of moisture). The soil paste shall then be transferred to the cylindrical trough and levelled from the top. The penetrometer will be so adjusted that the cone point just touches the surface of the sample. The pointer on the stem shall be adjusted to read zero or any other specific mark on the scale and then released to penetrate into the soil sample under its weight. The penetration shall be noted after about 30 seconds from the release of the cone. The moisture content of the sample shall be increased till a penetration of 1 in. is obtained. This moisture content when expressed as a percentage on the oven dry weight of the soil shall be taken as liquid limit. The liquid limit values can be more correctly determined by having about 3 readings with increasing moisture content and interpolating the corresponding value for one inch penetration from the graph.

FINAL TRIALS

In this part of the study, a large number of soils with wide range of liquid limits, from 20-85 which are almost extreme limits of soils, generally found in India, were selected and their liquid

TABLE 7

S. No.	Soil No.	Liquid limit per cent: Casagrande's mech. device	Liquid limit per cent: Penetrometer device
1	688	20.1	22.8
2	511	20.7	22.9
3	927	20.8	23.2
4	D IV	21.4	24.3
5	610	22.3	25.3
6	863	22.5	24.5
7	718	24.1	24.8
8	395	24.6	25.8
9	281	24.9	25.2
10	370	25.0	26.9
11	282	26.9	27.8
12	390	27.1	29.5
13	D I	28.0	28.3
14	423	28.2	30.0
15	D II	29.0	29.2
16	589	29.3	29.5
17	524	30.0	30.3
18	465	30.1	30.2
19	474	30.7	31.2
20	834	31.4	31.0
21	394	31.9	32.1
22	368	32.0	31.3
23	424	32.0	32.5
24	190	33.1	33.2
25	584	37.8	37.6
26	353	40.3	40.6
27	810	40.5	39.4
28	425	42.2	43.1
29	209	46.1	45.3
30	275	48.2	48.6
31	574	53.8	54.9
32	588	55.5	56.4
33	243	55.6	52.9
34	742	65.2	64.5
35	710	68.0	67.1
36	743	73.9	73.7
37	555	75.2	73.8
38	753	80.3	77.4
39	704	84.7	81.7
40	550	85.2	81.8

Note: Comparative values of some more samples are given in Table 8.

limits was determined both by the Casagrande's mechanical device and the proposed penetrometer according to the procedures laid down (Table 7).

The results clearly show that the liquid limit values obtained by both these methods fairly tally with each other for all soils, except when their liquid limit gets reduced below 25. In such cases, the results of penetrometer are somewhat higher than those obtained with mechanical device. It has already been shown that the latter device gives low results for such sandy soils. It was further considered to compare the results obtained by both these methods by adding varying percentages of graded sand to different soil samples. The plastic limit for each soil-sand mixture was also determined in the normal way and therefrom the results of plasticity index were calculated and are given in Table 8, Fig. 7.

The results show that the cone penetrometer gives more consistent results for a wider range of soils especially towards sandy side when compared to mechanical device. The decrease in plasticity index with the addition of sand is calculated from Hogentogler's* formula.

SIMPLIFICATION OF THE TEST PROCEDURE

To simplify the procedure of determining liquid limit by adjusting its moisture content to get one inch penetration it was considered to derive the same from a single reading of penetration at any moisture content. It has already been seen, vide Table 6 Fig. 6, that the two variables, penetration of the cone and the soil moisture, when under a fixed load, bear a linear relation provided the moisture content is close to its liquid limit and the

$$*P = \frac{P_1 S_1 X + P_2 S_2 Y + P_3 S_3 Z}{S_1 X + S_2 Y + S_3 Z}$$

Where X = % of sample A in mixture.
Y = do. B do.
Z = do. C do.
S_1 = do. of soil fines in sample A
S_2 = do. do. B
S_3 = do. do. C
P_1 = Plasticity index of soil fines in sample A.
P_2 = do. B.
P_3 = do. C.
P = do. the mixture.

TABLE 8

Comparative values of liquid limit and plasticity index as obtained by the proposed penetrometer and Casagrande's Mechanical Device after adding graded sand to soil samples.

Soil No.	Sand admixture per cent	Liquid limit per cent		Plastic limit per cent	Plasticity index — From liquid limit obtained by		Plasticity index calculated from Hogentogler's formula
		Penetrometer	Mechanical Device		Penetrometer	Mechanical Device	
Black cotton (JLG)	0	83.0	86.8	32.8	50.2	54.0	52.1*
	20	67.7	71.2	26.6	41.1	44.6	41.7
	40	55.0	56.1	21.2	33.8	34.9	31.3
	50	47.5	48.7	19.5	28.0	29.2	26.1
	60	41.3	41.9	18.4	22.9	23.5	20.8
	70	34.1	33.8	19.2	14.9	14.6	15.6
	80	28.5	25.6	19.8	8.7	5.8	10.4
	90	24.8	20.6	20.3	4.5	0.3	5.2
Soil No. 753	0	77.5	79.0	31.9	45.6	47.1	46.3*
	20	64.4	66.0	25.7	38.7	40.3	37.1
	40	50.0	50.6	21.4	28.6	29.2	27.8
	60	37.7	38.1	19.1	18.6	19.0	18.5
	80	26.8	23.6	18.7	8.1	4.9	9.3
	90	23.5	19.5	18.8	4.7	0.7	4.6
Soil No. 816	0	45.1	44.6	19.9	25.2	24.7	25.0*
	20	38.0	38.1	16.3	21.7	21.8	20.0
	40	31.6	31.5	15.7	15.9	15.8	15.0
	60	25.9	23.8	15.9	10.0	7.9	10.0
	70	23.8	20.5	16.3	7.5	4.2	7.5
	80	22.1	19.1	17.4	4.7	1.7	5.0
	85	22.1	18.1	17.8	4.3	0.3	3.8
DIII	0	34.5	35.0	19.3	15.2	15.7	15.4*
	20	29.1	29.5	16.9	12.2	12.6	12.3
	40	26.5	26.0	17.0	9.5	9.0	9.2
	60	24.2	20.4	17.7	6.5	2.7	6.2
	80	22.6	18.7	18.7	3.9	N. P.	3.1

* Mean value of plasticity index by both these methods.

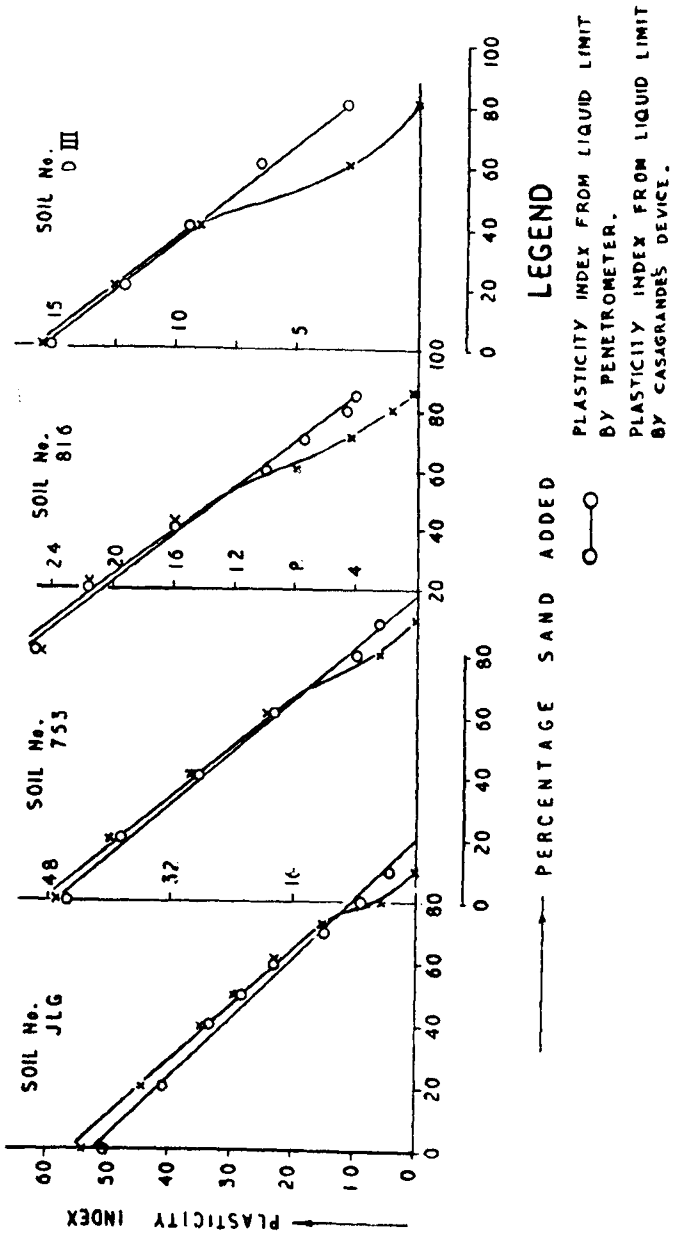


Fig. 7. Comparison of plasticity index results obtained from liquid limit values as determined by Casagrande's mechanical device and proposed penetrometer for soil sand mixtures

penetration ranges from 0.8 to 1.1 inches. These results were examined by the statistical section and after a careful study of the regression coefficients which depict the average change in the value of moisture content corresponding to a unit change in the value of penetration, the following relation has been worked out which can predict the value of liquid limit at 1 inch penetration with good accuracy.

$$\text{Liquid limit} = M\alpha + \tfrac{1}{4}(1-\alpha)\ (M\alpha + 15)$$

where α is the penetration of cone in inches

and $M\alpha$ is the moisture percentage corresponding to penetration of α inch.

It has further been verified that the estimated values of liquid limit calculated from this formula are very close to those actually determined. Comparative values are given in Table 9.

TABLE 9

Comparison of estimated values of liquid limit with those actually determined by the penetrometer

Soil No.	α in inches	$M\alpha$	Liquid limit per cent estimated	Liquid limit per cent determined
Base course soil	0.8	22.1	24.0	23.4
do.	1.10	24.0	23.0	23.4
524	0.82	28.4	30.3	30.2
do.	1.10	30.7	29.6	30.2
DIII/20S	0.85	29.4	31.0	31.5
do.	1.09	32.5	31.4	31.5
816/20S	0.86	38.0	39.8	39.7
do.	1.05	40.2	39.5	39.7
816	0.80	41.7	44.5	46.0
do.	0.91	44.5	45.8	46.0
do.	1.07	47.0	45.9	46.0
275	0.77	45.2	48.7	49.0
753/20S	0.82	61.5	64.9	65.6
do.	1.08	68.0	66.2	65.6
753	1.10	78.0	77.1	76.1

A multicurve chart as shown in Fig. 8 has also been developed from which moisture content at 1 in. penetration of the cone can be easily read, knowing moisture at any penetration between 0.8—1.1 in.

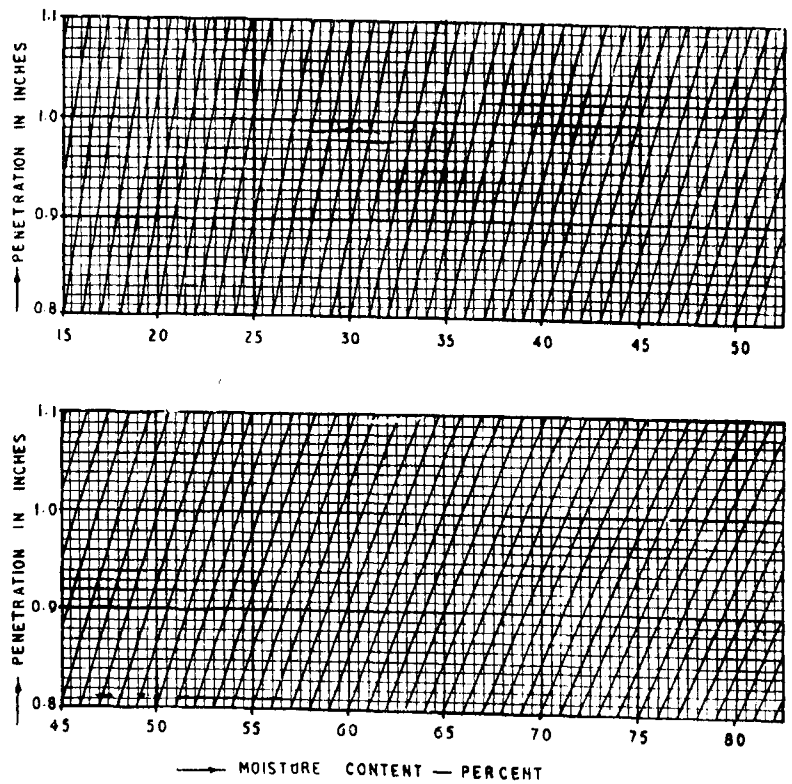


Fig. 8. **Multicurve chart for calculating liquid limit from penetrometer readings**

ESTIMATION OF SHEAR STRENGTH AT LIQUID LIMIT MOISTURE CONTENT AS GIVEN BY PENETROMETER

The theory of plastic flow, as involved in these cone penetration tests, has already been worked out by Kàrmàn, Prandtl and Hencky and the investigations of these Authors show that the hardness of plastic materials (resistance against penetration of a solid body into the material) is a simple multiple of the cube strength of the material. In turn, by definition, the hardness of plastic materials corresponds to the yield point because under a pressure greater than the hardness, the material flows. Hence, the yield point and the cube strength of plastic materials are connected with each other by a simple and well-defined relation.

This theoretical conclusion has been repeatedly verified by experiments. Thus, according to the theory, the resistance of a body with a long narrow cross section should be equal to 2.57 times the cube strength of the material, regardless of its hardness. The proposed cone of half angle of 15 deg.-30 min., at 1 in. penetration, on calculation gives 1.35 lb per sq. in. as its penetration resistance for a bearing area of 0.2416 sq. in. Further, in case of soils at liquid limit, the shear strength would only be about half of the cube strength i.e., compressive strength. The penetration by 1 in. of such cone in a soil would, therefore, amount to a shear strength of about $1.350/(2 \times 2.57) = 0.26$ lb per sq. in. which compares well with the estimated values of Skempton and Northey.

ACKNOWLEDGMENT

This work has been carried out as part of the research programme of the Central Road Research Institute, New Delhi, and the Authors are grateful for the interest taken by Shri S. R. Mehra, Director of the Institute, during the investigation.

REFERENCES

1. Atterberg, A. " Uber die Physikalische Bodenuntersuchung, und Uber die Plastizitat der Tone." Internationale Mitteilungen fur Bodenkunde, Vol. I, 1911.
2. Wintermyer, A.M. " Adoption of Atterberg Plasticity Tests for Subgrade Soils." Public Roads, Vol. 7, No. 6, August 1926.
3. Casagrande, A. " Research on the Atterberg Limits of Soils " Public Roads, Vol. 13, No. 8, October 1932.
4. " Standard Method of Determining the Liquid Limit of Soils ". AASHO. Designation T89-49. Standard Specifications for Highway Materials, Methods of Sampling and Testing, Part II, 6th edition, page 228. Published by the American Association of State Highway Officials, 1950.
5. British Standard 1377 : 1948 Page 20. Methods of Test for Soil Classification and Compaction.
6. A. S. T. M. Designation D423-54T. Tentative Method of Test for Liquid Limit of Soils.
7. Fundamentals of Soils Mechanics by Donald W. Taylor.
8. Soils Testing for Engineers by T. William Lambe.
9. A. W. Skempton & R. D. Northey, " The Sensitivity of Clays " Geotechnique, Vol. III, March 1952.

"SIZE AND QUANTITY OF AGGREGATES FOR SURFACE DRESSINGS"

By

C. G. Swaminathan*

AND

S. Bagchee*

CONTENTS

SYNOPSIS

A slight modification in the technique of surface dressing is suggested by spreading the aggregates for each coat, in two stages. The aggregates for the second course will be of a size smaller than that of the first course and, by wedging them into the voids, a well knit and interlocked surface could be obtained. Based on laboratory data and assumptions, the relative sizes and quantities have been worked out.

1. INTRODUCTION

1.1. The technique of surface dressing consists essentially of the proper embedment of a tightly packed stone course over a bituminous layer. For securing the bond between the aggregates and the binder, it is necessary that the stone course should comprise of aggregates laid in a single tier. If the stones are of mixed sizes as shown in Fig. 1 (wherein smaller size stones are spread along with the bigger sized aggregates), it is quite possible that, owing to the passage of traffic the bigger aggregates would get dislodged resulting in ravelling. Further, the smaller sizes

* Central Road Research Institute, New Delhi.

might get submerged in the bituminous binder and serve no purpose. If, on the other hand, the aggregates are of a single size uniformly as shown in Fig. 2, each stone being embeded in the layer of binder—then the above mentioned defects are not likely to arise. It is because of this that most of the practices abroad indicate the use of single sized aggregates for surface dressing.

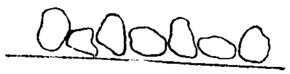


Fig 1.

Fig. 2.

Fig. 3.

1.2. A further development of this appears to be possible if the aggregates, shown in Fig. 2, are given additional interlocking support by smaller size chippings keyed into the top crevices of the first stone layer as shown in Fig. 3. In order that this structure may function effectively it is necessary that the two layers of aggregates will have to be laid separately. Also the space between any two aggregates should be such that the binder from below works upwards and also affords sufficient embedment for the smaller sized aggregates.

1.3. To achieve this effect it is obvious that the proper size and quantities of aggregates should be chosen for the two layers. The object of this note is to determine as to what should be the size and quantity of the stone chippings in order that a well knit, well embeded aggregate surface is presented to the traffic.

2. LABORATORY PROCEDURE FOR THE DETERMINATION OF THE SIZES AND QUANTITIES

2.1. In order to determine the relative sizes and quantities of aggregates required for a single coat (but consisting of two layers of stones), the procedure adopted was as under :

Selected, angular and irregular Delhi quartzite aggregates of the following five sizes were taken :

(a) Passing 1 in. and retained on $\frac{3}{4}$ in.

(b) Passing $\frac{3}{4}$ in. and retained on $\frac{1}{2}$ in.

(c) Passing $\frac{1}{2}$ in. and retained on 3/8 in.

(d) Passing 3/8 in. and retained on $\frac{1}{4}$ in.

(e) Passing $\frac{1}{4}$ in. and retained on 1/8 in.

These aggregates were chosen because of the fact that these are the normally employed sizes for bituminous surface dressings in India.

2.2. To determine the quantity of aggregates of a given size required to cover a unit area, the aggregates were hand-packed so as to have a shoulder to shoulder support between themselves over a given area of 10 in. $\times$ 8 in. The aggregates were then removed and weighed. Their bulk densities were also determined previously. Based on this density, the quantity of aggregates in cu. ft. required to cover a unit area is obtained. For two different values of the original bulk density of the aggregate, for each size, the quantities required for shoulder to shoulder packing were obtained. The figures are given in Table 1.

2.3. In order to determine the individual void spaces left after the aggregates are packed, a known quantity of the aggregates was taken and coated with a thin film of glycerine and dextrine paste. The aggregates were then placed in a tray of size 10 in. $\times$ 8 in., and lightly shaken so that the aggregates may all, more or less, lie in one layer. The tray also was previously coated with a thin film of glycerine and dextrine paste. Molten blown grade bitumen was then poured into the voids visible on the surface so that the aggregates are submerged until the tops of the aggregates are just visible, Photos 1 and 2, and the whole tray was then allowed to cool. On inverting the tray the whole block of the bitumen together with the embeded aggregate was easily taken out. The aggregates were then removed and an impression of the asphalt projections which in fact represent the voids, was taken. From this impression, the surface basal area of these voids was obtained with the help of a planimeter. It was noted here that the pattern of the basal surface area of the voids either on the top or at the bottom is more or less alike. The pieces of projecting solid asphalt which represent the voids were then cut with a knife so as to leave a very level base. These were then counted and sieved through a series of sieves, without exercising much pressure. The percentages of particles (by number) passing each sieve were obtained, 2.4. This procedure was adopted for each of the sizes menti,oned above, Photos 3, 4 and 5, except for the size passing $\frac{1}{4}$ in. and retained on 1/8 in., as it was difficult to get the individual asphalt pieces (representing the voids) without damaging them. The operation, however, was done for different rates of spread of the aggregates at intervals of 0.5 cu.ft. The number of different rates of spread taken for each size was such as to give one to two rates above and below the optimum quantity of aggregates required to give a single layer of shoulder to shoulder packing. The size of voids is given in Tables 2, 3, 4 and 5, and the surface area of the voids is given in Tables 6, 7, 8 and 9.

2.5. For obtaining the volume of voids when the aggregates are kept in one layer and packed very closely, a known quantity (Tables 6 to 9) of the aggregates was arranged in a previously weighed tray, and molten wax was poured into the voids of the one stone layer. On cooling, the wax was shaved off closely so as to yield as level surface as possible. The tray was again weighed and from the difference in weight the volume of the voids was obtained. The voids were also calculated based on the bulk volume of the aggregates used and the results are given in Table 10.

3. DISCUSSION OF THE RESULTS

3.1. Table 1 gives the quantities of aggregates of various sizes required to cover 100 sq. ft. area of the road surface, when the stones are packed shoulder to shoulder and to form a mat of one stone thickness. From this Table it can be seen that the quantity required decreases as the size decreases. It suggests that a little quantity of the small size chippings would suffice to act as a blotter and as such impair the proper embedment of the larger sized aggregates. Hence, it is obvious that in the aggregates used for surface dressing there should not be any appreciable amount of fines.

3.2. It can also be noted that whereas the quantities of aggregate in lbs per 100 sq. ft. for any one given size, for two determinations, are the same, the quantities in cu. ft. per 100 sq. ft. are widely different for the same size. This is as a result of the different sets of values of the bulk densities taken. Since the bulk densities for the first set of determinations (representing the loose bulk density) are lower than the corresponding bulk densities (representing the dense bulk density) for the second, the bulk volume of aggregates required is more for the first series than for the second. This stresses the importance of expressing the quantities in term of weight rather than in terms

of volume. Specifying the quantities in terms of volume is liable to give either an excessive or insufficient coverage. Hence it is suggested that specifications based on lb per unit area or area per unit weight of aggregate, similar to practices abroad, should be adopted in our country as well.

3.3. Tables 2, 3, 4, and 5 give the different sizes of the voids expressed as a percentage (by number) for the various sizes of aggregates taken and for different rates of spread. It can be seen from Tables 2, 3, and 4, that for aggregate size, viz. 1 in.-$\frac{3}{4}$ in., $\frac{1}{2}$ in.-$\frac{3}{4}$ in., and $\frac{1}{2}$ in.-$\frac{3}{8}$ in., nearly 15–30 per cent of the voids can be filled by the same size aggregates and that between 45–70 per cent of the voids can be filled by aggregates of the next lower size. It may be asked as to why the 15–30 per cent voids could not have been filled with the original aggregates themselves. The answer is that the voids were so irregular that, with the type of chipping that was used, it was not possible to fill up these voids with the same size aggregates although the volume of the voids as such indicated the possibility of wedging them.

3.4. Whereas, for the various rates of spread adopted for the sizes 1 in.-$\frac{3}{4}$ in. and $\frac{3}{4}$ in.-$\frac{1}{2}$ in., the layers of stones were of one stone thick for all the rates, for the sizes $\frac{1}{2}$ in.-$\frac{3}{8}$ in. and $\frac{3}{8}$ in.-$\frac{1}{4}$ in., the layer of stones for rates of spread above 2.5 cu.ft. for the former and 2 cu. ft. for the latter, was more than one stone thick. In the case of $\frac{3}{8}$ in.-$\frac{1}{4}$ in. aggregates, for rates of spread of 2.5 cu. ft. and above, the stones were in two tiers. It is because of this that for the $\frac{3}{8}$ in.-$\frac{1}{4}$ in. aggregate size, the percentage of voids (by number) having the same size as the aggregate is about 60 per cent whereas when the rate of spread is 2 cu. ft. the percentage of voids having the next lower size is 60 per cent.

However, from the various Tables it can safely be inferred that nearly over 70 per cent of the voids in the one stone layer for any size of the aggregate can be filled with aggregates of the next lower size. This leaves about 30 per cent of the voids to be filled by the still smaller size aggregates. Use of these smaller sizes would mean a gradation. Also the smaller size can be taken care of by the tolerances that are usually allowed while specifying the size of aggregates. Further, under the action of rollers some of the aggregates do get crushed. So, taking all these factors into consideration, it can safely be stipulated that all the voids in the one stone layer should be filled by aggregates of the next lower size.

3.5. Now that the size of the aggregates to fill the voids in the single stone layer is determined, the question arises as to what should be the relative quantities. The voids in a one stone layer are not always exactly similar to the voids when the aggregates are in a heap. Further, the filling up of the voids in the one stone layer has to be done by laying another layer of the aggregates, so that they may wedge into the voids of the first layer. For this, Tables 6, 7, 8 and 9 are drawn up, giving the surface area of the voids in the one stone layer. It can be seen from these Tables that the surface basal area of the voids goes on decreasing as the quantity of aggregate goes on increasing until the aggregates are packed shoulder to shoulder. If the aggregates are laid in more than one layer, then the area as measured with the planimeter tends to be erratic. However, it can be seen from the Tables that when the aggregates are closely packed to form a single layer, the area of the surface voids is about 35–40 per cent of the total area taken for coverage.

Based on this, the relative quantities of the aggregates can be worked out. For example, it is seen from Table 1 that 380 lbs of $\frac{3}{4}$ in. – $\frac{1}{2}$ in. aggregate is required to cover 100 sq. ft. Since only 60 per cent of this area is occupied by the solid aggregate, the remaining 40 per cent (allowing for whip off and tolerance) of the area is to be filled by aggregate of the next lower size, *viz.*, $\frac{1}{2}$ in. – 3/8 in. The quantities of aggregate of this size required to cover 100 sq. ft. is 233.9 lb (Table 1). But 233 lb would actualy cover 60 sq. ft. So the quantity of $\frac{1}{2}$ in. – $\frac{3}{8}$ in. aggregate required to cover the 40 sq. ft. voids area in the single stone layer composed of $\frac{3}{4}$ in. – $\frac{1}{2}$ in. aggregate is $40/60 \times 233.9$ lb $= 155.7$ lb. Empirically, it can be said that the quantity of aggregates of the next size required to fill up the top voids of a closely packed stone layer in a unit area is equal to 2/3 its weight to cover the same unit area.

It may be that very close packing may result in the possibility of the aggregate (especially if the aggregates are not very hard or tough) getting crushed. Also a very dense packing of the first course may not allow the binder to embed, partially at least, the aggregates of the second course. So it may be considered advisable to adopt about 0.5 cu.ft. less than the optimum quantity of aggregates required for dense packing. Based on this, Table 12 is worked out giving the range of limits for the quantities of aggregates to be adopted for the various sizes.

3.6. For Tables 6, 7, 8 and 9 the percentage void volume was obtained by the equation :

$$V = \frac{80 \times 16.39X - \frac{W}{g}}{80 \times 16.39X} \times 100$$

where X = average mat thickness.

W = weight of aggregate to cover 80 sq. in. in gms.

g = specific gravity of the stone.

16.39 = conversion factor for metric units.

From Table 6 it can be seen that the void percentage goes on decreasing as the quantity of aggregates is increased, until the stones are packed shoulder to shoulder to form a one stone layer. If the quantity of aggregates laid is more than the optimum, the void volume tends to decrease further, Tables 8 and 9, but in these cases the void volume is that obtained for a two stone layer.

From Tables 6 to 9 it can be noticed that the percentage void volume in a tightly packed one stone layer is about 60 per cent for the various sizes chosen.

But, the void volume as obtained by actual measurement with wax, as given in Table 10, gives a different picture. Figures in column 6 of Table 10 are obtained from the equation :

$$V = \frac{\text{Vol. of voids (Column 5)}}{w/2.64 + \text{volume of voids}} \times 100$$

Where w = weight of aggregate to cover 80 sq. in. (in gms.)

Similarly figures in column 8 of Table 10 were calculated from the equation :

$$V = \frac{\text{Bulk volume as obtained from Col. 3 or 7} - W/g}{\text{Bulk volume}} \times 100$$

It can be seen that the percentage of voids so obtained varies from 40–50 per cent. It is of interest to note that the percentage volume for a closely packed sphere also is about 47.5 per cent. Hanson[3] has assumed that the volume of voids in a single stone layer is about 50 per cent. However, it should be remembered that computations based on void volume measurements are liable to be erratic.

In the one stone layer, it was found that the volume of the surface area void pattern was similar in both the top and bottom of the experimental blocks. So, it can be assumed that half of this void volume is below the centre line of the aggregates. The bottom portion will be filled by the rising bitumen, whereas the top voids should be filled by the smaller size aggregates.

Taking the figures from Table 12, it is seen that 5 cu. ft. of 1 in. – $\frac{3}{4}$ in. size aggregates are required for one stone layer; 25 per cent of this volume is due to the top voids and it has to be filled by the smaller size aggregates. This means that 1.25 cu. ft. of volume should be filled by aggregate of the next lower size namely the $\frac{3}{4}$ in. – $\frac{1}{2}$ in. aggregate. Now, on the assumption that 10 cu. ft. of aggregate (of any size) when laid shoulder to shoulder in a single layer will have only 5 cu. ft. of solid volume, the quantity of aggregate required to fill up the 1.25 cu. ft. volume of voids in the first stone layer amounts to 2.5 cu. ft.

This assumption may not be valid since, when the 1.25 cu. ft. is filled by the aggregates, the aggregates touching one another will not be of the same size. However, the quantities of the aggregates of the next lower size required to fill the top voids of the first stone layer (based on volume) for the various sizes and for various quantities (obtained from Table 12) are worked out in Table 11.

From Table 11 it can be seen that the quantity of aggregates of the next lower size required to fill up the voids of the first stone layer, as obtained by the volumetric method, is more or less similar to that obtained by the surface area method. Yet the quantity obtained by the volume basis cannot be relied upon completely because of the assumptions involved. Also, the volume of voids in a one stone layer will not be the same, after compaction by roller, as it was at the time of laying. But the surface area of the voids is likely to be more or less the same. Further, since surface dressing is essentially a single layer construction, computation of the quantities of the aggregates based on surface area is better suited than that based on the volume of voids.

3.7. It may be argued that the quantities given in Table 12 may be too rigorous to adopt, since the sizes specified are of single size. But when the figures are compared with the tolerances that have been suggested for single sized stones by the British Standard[1] and those that are being proposed to be adopted by the I.R.C.[2], it is seen that the specifications given in Table 12 are workable. According to the recommendations for the specifications of the single size, it is proposed that not more than 15 per cent by weight of the aggregate should belong to the next bigger size and that not more than 7 per cent by weight should belong to the undersize. This means that about 78 per cent of the aggregates should fall within a specified sieve range.

In order to lay a single coat as per specification in Table 12 it is seen that about 337-380 lb per 100 sq. ft. of $\frac{3}{4}$ in. to $\frac{1}{2}$ in. size

Photo 1. **Aggregates embedded in bituminous matrix**

Photo 2. **Voids left after the removal of aggregates**

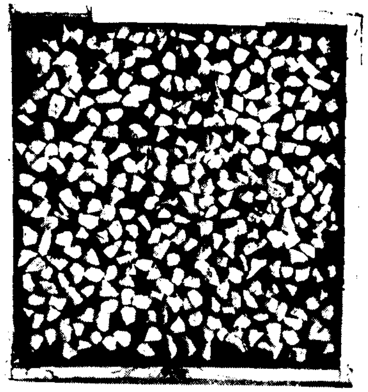


Photo 3.

Laid with 1.5 cu. ft. per 100 sq. ft. of aggregate of a size passing ½ in. and retained on ⅜ in. sieve

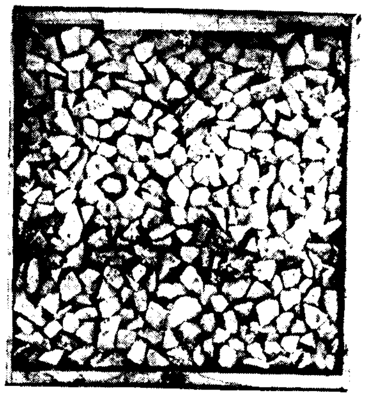


Photo 4.

Laid with 3.75 cu. ft. of aggregate per 100 sq. ft. of a size passing ¾ in. and retained on ½ in. sieve

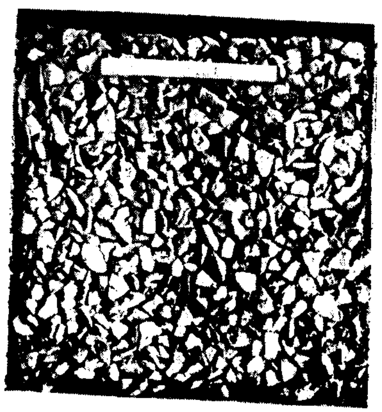


Photo 5.

Laid with 3.75 cu. ft. per 100 sq. ft. of aggregate passing ¾ in. and retained on ½ in. sieve followed by 1.5 cu. ft. per 100 sq. ft. of aggregate of a size passing ½ in. and retained on ⅜ in. sieve

aggregates are required together with 124-156 lb of ½ in.-3/8 in. size aggregates. According to the requirements of the single size stone proposed by the I.R.C.[2] and specified by the B.S.S.[1], it is noted that out of 100 lb of the ¾ in.-½ in. aggregates about 15 lb would be of size 1 in.-¾ in., 65 lb of size ¾ in.-½ in. and 13 lb of ½ in.-3/8 in. Similarly, out of 100 lb of the ½ in.-3/8 in. aggregates about 15 lb would be of size ¾ in.-½ in., 55 lb of size ½ in.-3/8 in. and 23 lb of 3/8 in.-¼ in. If the very small sizes and fines are neglected, 400 lb of the bigger size stones would yield about 60 lb of 1 in.-¾ in. aggregates, 270 lb of ¾ in.-½ in. and 52 lb of ½ in.-3/8 in. size. Also 150 lb of the smaller size stone would yield about 22.5 lb of ¾ in.-½ in. aggregate, 82.5 lb of ½ in.-3/8 in. Since the fines are injurious in as much as they act as blotters and since the presence of a few bigger size aggregates would only slightly lower the covering capacity of the aggregate, based on the tolerance limitations, 400 lb of the bigger size and 150 lb of the next smaller size would yield 352.5 lb of aggregates retained on ½ in. and 134.5 lb of aggregates ½ in.-3/8 in. These are well within the limits specified for *B* in Table 12. Similarly for the other sizes also the quantities required could be so proportioned so as to fall within the recommendations given.

3.8. It may be thought that the cost of construction of the single coat surface dressing would increase because of the following reasons:

(a) Aggregates have to be sieved to yield the two different sizes.

(b) Aggregates have to be spread in two layers.

(c) Rolling has to be done in two stages.

From a purely academic point of view it is seen that there should not be much of a difference in the costs, for, if the aggregates are supplied to satisfy the tolerance limits set forth there would not be need for any sieving. Secondly, there is no substantial increase in the quantities of aggregates required, for, whereas the present practice for single coat surface dressing stipulated 5.5 cu. ft. to 6 cu. ft. of standard ½ in. (passing ¾ in. and retained on 3/8 in.) size aggregate, the recommendation given in this Paper suggests a total quantity of about 5.25 cu. ft. to 6 cu. ft. of aggregates. So the cost involved in spreading the aggregates also cannot be very appreciably different. Further, if according to the present practice the number of passages of the roller is about a dozen times, according to the procedure recommended, the roller will have to pass twice on the first layer of aggregates and about 10 times after the second layer of aggregates is spread. So the total cost involved should more or less be the same. However, a rough estimate of the cost involved shows that by laying the aggregates in two layers and by adopting a two stage rolling operation the extra cost involved is about as. 12 per 100 sq. ft., Appendix 1. Considering the fact that a well knit surface is obtained, this extra cost is indeed negligible.

3.9. The various quantities and sizes given in Table 12 are essentially for the single coat surface dressing. For exmaple, for a single coat surface dressing using $\frac{3}{4}$ in.-$\frac{1}{2}$ in. and $\frac{1}{2}$ in.-3/8 in. aggregate, it is suggested that after spraying of sufficient binder, about 337-380 lb of $\frac{3}{4}$ in.-$\frac{1}{2}$ in. aggregate should be spread evenly and lightly rolled once to embed the aggregates. Immediately about 124-156 lb of $\frac{1}{2}$ in.-3/8 in. aggregates should be spread evenly, brushed lightly and then rolled completely. If a second coat of surface dressing is to be given over this, the second coat of binder should be sprayed and then aggregates of $\frac{1}{2}$ in.-3/8 in. followed by 3/8 in.-$\frac{1}{4}$ in. or aggregates of 3/8 in.-$\frac{1}{4}$ in. followed by $\frac{1}{4}$ in.-1/8 in. used. Similarly, various sizes could be chosen depending on the size of the stones for the first coat.

4. CONCLUSIONS

(1) It is suggested that a well knit single coat surface dressing can be obtained by laying a uniform size single stone layer to be followed by a smaller size aggregate to fill in the top voids in the surface and to give additional wedge support to the first stone course.

(2) It is suggested that specifications for the quantities of aggregates to be employed in surface dressings should be expressed in terms of weight rather than by volume.

(3) Based on the assumptions that nearly all the surface voids in a single stone layer can be filled by aggregates of the next smaller size and that the area of the surface voids in a single stone layer is about 40 per cent of the overall covered area, the relative sizes and quantities required for a single coat and a two coat surface dressing can be worked out.

ACKNOWLEDGMENT

The Authors wish to thank Shri K. P. Bannerji, Senior Laboratory Assistant for his help in collecting the data.

The Authors also wish to thank the Director, Central Road Research Institute for his encouragement and permission to publish this Paper.

REFERENCES

1. British Standard Specifications for Single Size Road-stones and Chippings, BSS-63 of 1951.
2. Proposed Tolerances for Standard Size Road Stones. Specifications and Standards Committee of the Indian Roads Congress.
3. Cambell - The Use of Asphaltic Materials with Particular Reference to Ceylon. Engineering Association of Ceylon — 1951.

TABLE 1

Quantity of aggregates required for one layer thickness of stone when packed shoulder to shoulder

No.	Size of aggregates	Quantity in cu. ft. per 100 sq.ft.	Bulk density in lb. per cu. ft.	Quantity in lb per 100 sq. ft.
1	Passing 1 in. retained $\frac{3}{4}$ in.	(i) 5.47 (ii) 4.85	85.07 96.0	465.8 465.8
2	Passing $\frac{3}{4}$ in. retained $\frac{1}{2}$ in.	(i) 4.44 (ii) 3.94	85.7 96.5	380.6 380.6
3	Passing $\frac{1}{2}$ in. retained $\frac{3}{8}$ in.	(i) 2.86 (ii) 2.51	81.75 93.0	233.9 233.9
4	Passing $\frac{3}{8}$ in. retained $\frac{1}{4}$ in.	(i) 2.32 (ii) 2.06	80.59 91.0	187.3 187.3
5	Passing $\frac{1}{4}$ in. retained $\frac{1}{8}$ in.	(i) 1.35 (ii) 1.24	79.0 86.0	107.0 107.0

TABLE 2

Percentage (by number) of the size of voids in the aggregates. Aggregate size—passing 1 in. - retained 3/4 in.

Size of voids	Rate of spread of aggregates in cu. ft. per 100 sq. ft.			
	3.0 cu. ft.	3.5 cu. ft.	4.0 cu. ft.	4.5 cu. ft.
$1\frac{1}{2}$ in.-$1\frac{1}{4}$ in.	5.0	4.8		
$1\frac{1}{4}$ in.-1 in.	12.9	13.0	—	—
1 in.-$\frac{3}{4}$ in.	27.4	27.5	—	—
$\frac{3}{4}$ in.-$\frac{1}{2}$ in.	46.7	46.7	14.5	18.8
$\frac{1}{2}$ in.-$\frac{3}{8}$ in.	8.0	8.0	44.5	68.0
$\frac{3}{8}$ in.-$\frac{1}{4}$ in.	—	—	28.0	11.4
			13.0	1.8

TABLE 3

Percentage (by number) of the size of voids in the aggregates. Aggregate size - passing 3/4 in.-retained 1/2 in.

No.	Size of voids	Rate of spread of aggregates in cu. ft. per 100 sq. ft.			
		3.0 cu. ft.	3.5 cu. ft.	4.0 cu. ft.	4.5 cu. ft.
1	1 in. - $\frac{3}{4}$ in.	1.5	0.9	1.2	0.6
2	$\frac{3}{4}$ in. - $\frac{1}{2}$ in.	29.3	18.4	20.4	28.4
3	$\frac{1}{2}$ in. - $\frac{3}{8}$ in.	40.8	50.0	54.5	50.3
4	$\frac{3}{8}$ in. - $\frac{1}{4}$ in.	22.2	30.7	23.9	20.7
5	$\frac{1}{4}$ in. - $\frac{1}{10}$ in.	6.2	—	—	—

TABLE 4

Percentage (by number) of the size of voids in the aggregates. Aggregate size-passing 1/2 in. - retained 3/8 in.

No.	Size of voids	Rate of spread of aggregates in cu. ft. per 100 sq. ft.			
		2.5 cu. ft.	3.0 cu. ft.	3.5 cu. ft.	4.0 cu. ft.
1	$\frac{3}{4}$ in. - $\frac{1}{2}$ in.	10.2	2.7	1.3	2.9
2	$\frac{1}{2}$ in. - $\frac{3}{8}$ in.	26.5	20.0	15.9	17.4
3	$\frac{3}{8}$ in. - $\frac{1}{4}$ in.	51.8	66.6	69.1	67.7
4	$\frac{1}{4}$ in. - $\frac{1}{10}$ in.	11.5	10.7	13.7	12.0

TABLE 5

Percentage (by number) of the size of voids in the aggregates. Aggregate size - passing 3/8 in.-retained 1/4 in.

No.	Size of voids	Rate of spread of aggregates in cu. ft. per 100 sq. ft.			
		2.0 cu. ft.	2.5 cu. ft.	3.0 cu. ft.	3.5 cu. ft.
1	$\frac{1}{2}$ in. - $\frac{3}{8}$ in.	2.2	7.2	9.5	21.0
2	$\frac{3}{8}$ in. - $\frac{1}{4}$ in.	37.5	60.3	57.8	71.0
3	$\frac{1}{4}$ in. - $\frac{1}{10}$ in.	60.3	32.5	32.7	8.0

TABLE 6

Volume of voids and surface area of voids in aggregates

Aggregate size — P 1 in. - R 3/4 in.

Aggregate by volume to cover 100 sq. ft. cu. ft.	Aggregate by weight to cover 80 sq. in. or 516.1 sq. cm. assuming bulk density = 96 lb per cu. ft.	Voids (volume) calculated in cc. assuming mat thickness as $\frac{7}{8}$ in. and specific gravity of aggregate as 2.64	Per cent voids (volume) in mat	Surface area of the voids measured in sq. cm.	Per cent surface area of the voids
3.0	726 gms.	872.1	76.0	295.5	57.2
3.5	847 gms.	826.4	72.1	240.5	46.5
4.0	968 gms.	780.7	68.1	210.4	40.7
4.5	1089 gms.	735.1	64.1	183.2	35.4
Single layer shoulder to shoulder pack 4.85	1175 gms.	703.7	61.4	—	—

TABLE 7

Volume of voids and surface area of voids in aggregates

Aggregate size — P-3/4 in. — R-1/2 in.

Aggregate by vol. to cover 100 sq. ft. cu. ft.	Aggregate by wt. to cover 80 sq. in. or 516.1 sq. cm. assuming bulk density = 96.5 lb per cu. ft.	Voids (volume) calculated in cc. assuming mat thickness as $\frac{5}{8}$ in. and specific gravity of aggregate as 2.64	Per cent voids (volume) in mat	Surface area of the voids measured in sq. cm.	Per cent surface area of the voids
3.0	730 gms.	543.9	66.4	251.7	48.8
3.5	851 gms.	498.2	60.8	177.3	34.4
4.0	973 gms.	480.9	59.7	182.0	35.3
4.5	1097 gms.	405.4	49.5	153.6	29.8
Single layer shoulder to shoulder pack 3.94	960 gms.	457.1	55.7	—	—

TABLE 8

Volume of voids and surface area of voids in aggregates

Aggregate size — P 1/2-in. - R 3/8-in.

Aggregate by volume to cover 100 sq. ft. cu. ft.	Aggregate by weight to cover 80 sq. in. or 516.1 sq. cm. assuming bulk density = 93 lb per cu. ft.	Voids (volume) calculated in cc. assuming mat thickness as 7/16 in. and specific gravity of aggregate as 2.64	Per cent voids (volume) in mat	Surface area of the voids measured in sq. cm.	Per cent surface area of the voids
2.5	586 gms.	352.4	61.4	198.7	38.5
3.0	703 gms.	308.3	53.8	221.2	42.9
3.5	820 gms.	264.1	46.0	244.5	47.4
4.0	937 gms.	220.0	38.4	218.5	42.3
Single layer shoulder to shoulder pack 2.51	590 gms.	352.4	61.4	—	—

TABLE 9

Volume of voids and surface area of voids in aggregates

Aggregate size — P-3/8 in. — R-1/4 in.

Aggregate by vol. to cover 100 sq. ft. cu. ft.	Aggregate by wt. to cover 80 sq. in. or 516.1 sq. cm. assuming bulk density = 86 lb per cu. ft.	Voids (volume) calculated in cc. assuming that thickness as 5/16 in. and specific gravity of aggregate as 2.64	Per cent voids (volume) in mat	Surface area of the voids measured in sq. cm.	Per cent surface area of the voids
2.0	458 gms.	236.9	57.8	181.0	35.1
2.5	573 gms.	185.9	45.4	209.0	40.5
3.0	688 gms.	150.1	36.6	265.5	51.4
3.5	803 gms.	106.7	26.0	284.5	55.1
Single layer shoulder to shoulder pack 2.05	472.5 gms.	231.4	56.4	—	—

TABLE 10

Volume of voids (obtained and as calculated) in one stone layer packed shoulder to shoulder

No.	Size	Aggregate vol. per 100 sq. ft.	Wt. taken for 80 sq. ft.	Voids as obtained with wax		Voids calculated	
				Vol. of voids	Percentage of voids	Bulk vol.	Percentage of voids
1	1 in. — ¾ in.	4.85 cu. ft.	1175 gms.	405 cc.	47.6	763.1 cc.	41.6
2	¾ in. — ½ in.	3.94 cu. ft.	960 gms.	305 cc.	45.6	619.9 cc.	41.3
3	½ in. — ⅜ in.	2.51 cu. ft.	590 gms.	237 cc.	51.4	395.0 cc.	43.4
4	⅜ in. — ¼ in.	2.05 cu. ft.	472 gms.	191 cc.	51.6	322.5 cc.	44.5

(Specific gravity of stone=2.64)

TABLE 11

Quantity of aggregates for the second layer based on volume of voids

No.	Size for the first layer	Quantity in cu. ft. per 100 sq. ft.	Size of aggregates for second layer	Quantity in cu. ft. per 100 sq. ft. = ½ the quantity in column 3
1	1 in. — ¾ in.	4.25–5.0	¾ in. — ½ in.	2.12–2.5
2	¾ in. — ½ in.	3.75–4.25	½ in. — ⅜ in.	1.87–2.12
3	½ in. — ⅜ in.	2.20–2.74	⅜ in. — ¼ in.	1.10–1.37
4	⅜ in. — ¼ in.	2.00–2.32	¼ in. — ⅛ in.	1.0–1.16

TABLE 12

Suggested quantity and size of aggregates for a single coat surface dressing with two layers of aggregates

No.	First Layer				Second Layer				Nomenclature
	Stone size	Wt. in lb per 100 sq. ft.	Assumed bulk density in lb per cu. ft.	Quantity in cu. ft. per 100 sq. ft.	Stone size	Wt. in lb per 100 sq. ft.	Assumed bulk density in lb per cu. ft.	Quantity in cu. ft. per 100 sq. ft.	
1	1 in.–¾ in.	405–450	90	4.50–5.0	¾ in.–½ in.	225–253	90	2.5–2.8	A
2	¾ in.–½ in.	337–380	90	3.75–4.25	½ in.–⅜ in.	124–156	85	1.5–1.8	B
3	½ in.–⅜ in.	186–233	85	2.20–2.75	⅜ in.–¼ in.	107–124	80	1.3–1.5	C
4	⅜ in.–¼ in.	161–186	80	2.00–2.32	¼ in.–⅛ in.	71	80	0.89	D
5	¼ in.–⅛ in.	107	80	1.34	—	—	—	—	E

Appendix 1

Difference in Costs (Rough Estimate).

The difference in costs between the conventional method and the suggested procedure for laying the aggregates and rolling them in two layers for a single coat surface dressing will be as under.

Assuming that the same quantity of binder and aggregates (say 5.25 cu. ft. per 100 sq. ft.) is adopted for both the cases, the extra cost for the two stage construction will be due to

(a) Sieving of the aggregates.

(b) Extra rolling.

(c) Extra cost for the material of different sizes. Since the quantity of aggregates is the same, extra cost for spreading will be negligible.

(i) Assuming that two coolies (at Rs 1–12 per coolie per 8 hour day) take one hour to sieve this quantity, cost for sieving will be $\frac{2\times 1.75}{8.0}=0.44$ rupees.

(ii) Cost of extra rolling, hire charges of roller + fuel + labour for rolling is assumed as Rs 60 per 8 hour day. Rolling time for normal rolling is assumed at $\frac{1}{2}$ hour for 800 sq. ft., since the first layer of stones need only to be embedded, extra cost for the first course would cost $60\times\frac{1}{8}\times\frac{1}{8}\times\frac{1}{2}\times\frac{1}{2}=0.24$ rupees.

(iii) Assuming the cost of $\frac{1}{2}$ in. gauge and $\frac{3}{8}$ in. gauge metal are Rs 33 and Rs 34 respectively per 100 cu.ft. and on the basis that in one case the 5.25 cu. ft. of metal is $\frac{1}{2}$ in. gauge and the other 5.25 cu. ft. of $\frac{1}{2}$ in. and 1.5 cu. ft. of $\frac{3}{8}$ in., the extra cost would be $\frac{33\times 15}{100\times 4}+\frac{34\times 3}{200}-\frac{33\times 5.25}{100}=0.02$ rupees per 100 sq. ft.

So the total increase in cost $=0.44+0.24+0.02$

$=$ Rs 0.70 per 100 sq. ft.

$=$ say 12 annas per 100 sq. ft.

"SOME LABORATORY INVESTIGATIONS FOR IMPROVING BLACK COTTON SOIL IN ROAD CONSTRUCTION"

By

S. R. MEHRA & L. R. CHADDA

INTRODUCTION

Black cotton soil commonly known as "Regur" occurs in large tracts of India. This soil usually possesses high base exchange capacity and contains a large percentage of colloidal clay particles which gives it excessive shrinkage and expansive properties and reduces its load bearing capacity to extremely low figures in the presence of moisture. These properties make black cotton soil a most unsuitable material either for use as such for engineering structures or for supporting the same as a base, a subgrade or a foundation. This study is confined to the possible use of such soils as base course for roads.

For purposes of road construction, black cotton soil has been held unsuitable on account of exhibiting abnormal shrinkage in the dry state and excessive softening in the wet condition. Its compressive strength in the dry state is very high, and ranges from 500-600 lb per sq. inch. In the rainy season, however, the soil after absorption of water becomes so soft that it is unable to support even light traffic, unless an extremely thick pavement is provided. It also swells up under pavement, with the absorption of water through capillarity and is likely to exert a great upward pressure, resulting in damage to the pavement. But, when the water evaporates in the dry season the soil shrinks abnormally and forms wide cracks which often extend to considerable depth. This typical behaviour of black cotton soil under different climatic conditions make the problem of road construction in such areas not only difficult but also very expensive.

OBJECT OF THE STUDY

Attempts have been made in the B & R Research Laboratory, Karnal, to find out the most economical method of stabilizing black cotton as a base course for road construction purposes.

The obvious method of stabilizing such a soil would be to add the requisite quantity of sand to it, to improve its grading. But the availability of sand within economic distance and the pulverization of the hard soil at a reasonable rate are practical difficulties. Experiments were, therefore, undertaken to study alternative methods making use of admixtures like lime, cement, gypsum etc., for stabilizing black cotton soil. Results obtained are presented in this note. Physical characteristics of eight soils selected for the study are given in Table 1.

TABLE 1

Physical characteristics of black cotton soils selected for the study

Sl. No.	Liquid Limit	Plasticity Index	Sand per cent	Total soluble solids	*pH*	Exchangeable Bases *Na & K* m.e.	Exchangeable Bases *Ca* m.e.	Base Exchange capacity in m.e.	Clay per cent
1	60.5	30.1	9.2	0.17	9.42	2.0	35.5	41.40	36.20
2	65.7	35.0	10.6	0.11	9.60	2.8	34.5	39.80	40.30
3	68.8	38.5	9.6	0.47	9.20	Nil	34.6	38.20	42.60
4	88.61	54.26	4.9	0.28	8.75	19.38	27.10	—	—
5	66.21	34.2	13.5	0.23	8.50	12.55	35.45	48.88	52.0
6	80.87	42.41	5.90	0.22	8.50	27.60	20.40	47.30	45.80
7	72.27	33.99	5.00	0.16	8.40	8.50	58.60	60.54	43.76
8	88.23	52.40	2.20	2.39	8.15	27.94	21.45	49.94	55.72

EFFECT OF LIME

For high density the compaction of soils is best done at optimum moisture. The effect of increasing concentrations of lime on the optimum moisture of all the soils selected was studied and the results obtained are given in Table 2.

TABLE 2

Effect of increasing concentrations of hydrated lime on the optimum moisture of black cotton soils

Soil No.	Opitimum moisture with increasing percentage of lime: Blank	2.5 per cent	5.0 per cent	7.5 per cent	10.0 per cent	15.0 per cent	20.0 per cent	25.0 per cent
1	16.0	16.0	16.5	16.5	17.0	18.0	19.0	20.0
2	17.0	17.0	17.0	17.5	18.0	18.5	19.5	20.5
3	16.0	16.0	16.0	16.0	17.0	18.0	19.0	19.5
4	17.0	18.0	19.0	20.0	21.0	22.0	23.0	24.0
5	16.0	16.3	16.5	16.8	17.0	17.0	17.0	17.0
6	21.0	21.0	21.0	21.0	21.0	22.0	23.0	24.0
7	19.0	20.2	21.0	22.0	23.0	23.6	24.3	25.0
8	20.0	20.2	20.4	20.7	21.0	22.8	24.4	26.0

As the optimum moisture rises with increase in percentage of lime it is necessary that for construction purposes, optimum moisture should be determined after addition of lime.

The effect of adding lime in increasing concentration on the following physical characteristics of resultant lime-soil mass was studied.

1. Volumetric shrinkage
2. Resistance to penetration
   (i) Immediately on addition of lime
   (ii) After allowing it to set
3. Moisture absorption
4. Compressive strength

TABLE 3

Effect of lime on the volumetric shrinkage of black cotton soil

Lime added per cent	Soil No. 1		Soil No. 3		Soil No. 4		Soil No. 5		Soil No. 6		Soil No. 7		Soil No. 8	
	Density	Shrinkage per cent	Density	Shrinkage per cent	Density	Shrinkage per cent	Density	Shrinkage per cent	Density	Shrinkage per cent	Density	Shrinkage per cent	Density	Shrinkage per cent
0	—	24.78	2.23	27.76	1.887	14.38	—	15.52	1.801	11.8	1.798	17.03	1.758	16.62
2.5	1.84	13.06	—	—	1.818	12.30	1.827	7.15	1.781	9.82	1.737	13.78	1.772	13.37
5.0	1.78	8.41	1.807	12.15	1.803	10.04	1.772	6.14	1.690	7.65	1.642	10.86	1.842	12.49
7.5	1.697	7.36	1.857	10.48	1.716	9.54	1.726	5.38	1.696	6.68	1.646	9.06	1.727	12.14
10.0	1.653	5.98	1.738	8.88	1.765	9.29	1.692	4.85	1.660	5.98	1.583	8.74	1.763	10.62
15.0	—	—	1.630	7.13	1.602	6.14	1.629	3.35	1.591	3.93	1.523	6.58	1.690	8.34
20.0	—	—	1.59	6.22	1.536	5.02	1.565	0.79	1.573	3.73	1.495	5.36	1.600	4.97
25.0	—	—	1.53	3.51	1.548	4.20	1.518	0.40	1.526	2.38	1.446	3.60	1.535	3.24

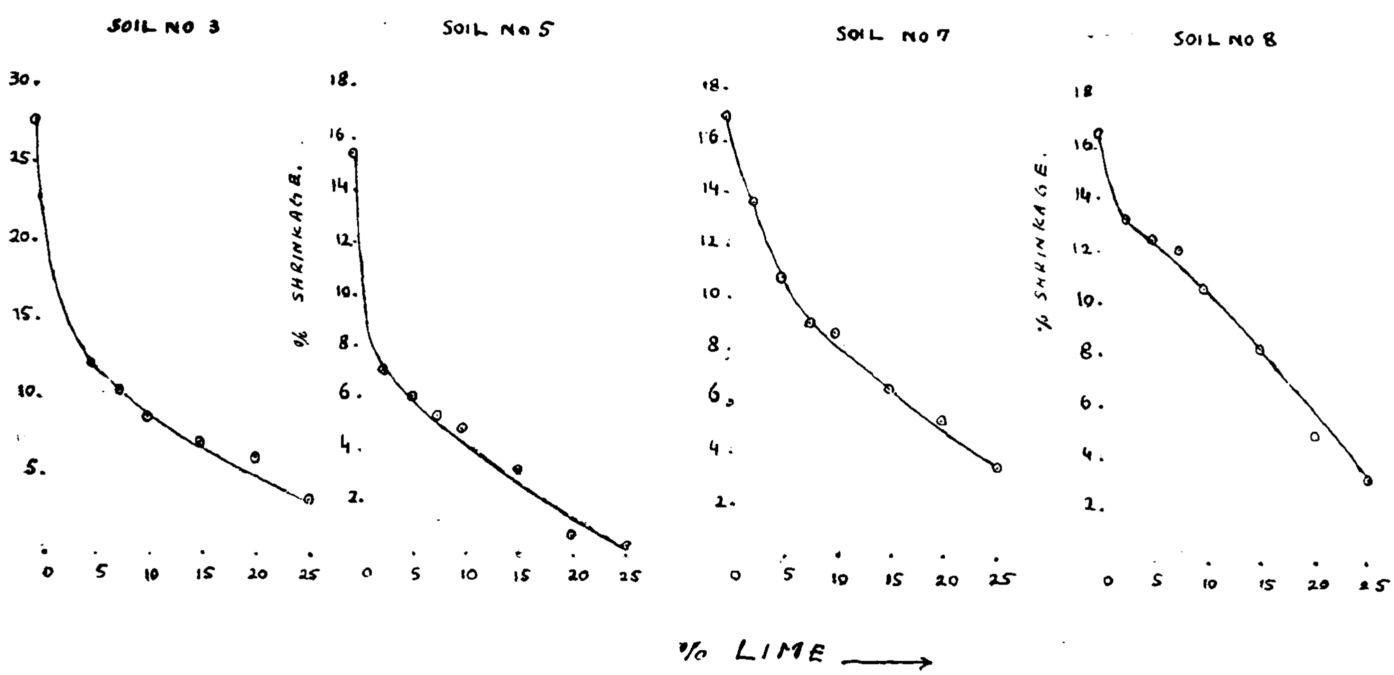


Fig. 1. **Effect of lime on the volumetric shrinkage of black cotton soil**

Volumetric shrinkage

Some of the soils mentioned in Table 1 were compacted with increasing percentage of hydrated lime at the optimum moisture. The volume of compacted blocks was determined at the time of compaction and after curing for seven days in humid air with subsequent drying at 50°C to constant weight. Results obtained are given in Table 3 and reproduced in Fig. 1.

It will appear from Table 3 that the shrinkage of compacted soil appreciably decreases with the addition of lime.

Resistance to penetration

Resistance to penetration of lime-soil compacted blocks was determined with Proctor's needle both (i) immediately after the addition of lime, and (ii) after allowing it to set.

(i) Under the first condition, the resistance to penetration was determined not only with increasing percentage of lime, but also at different moistures of compaction. Results obtained are given in Table 4 and reproduced in Fig. 2.

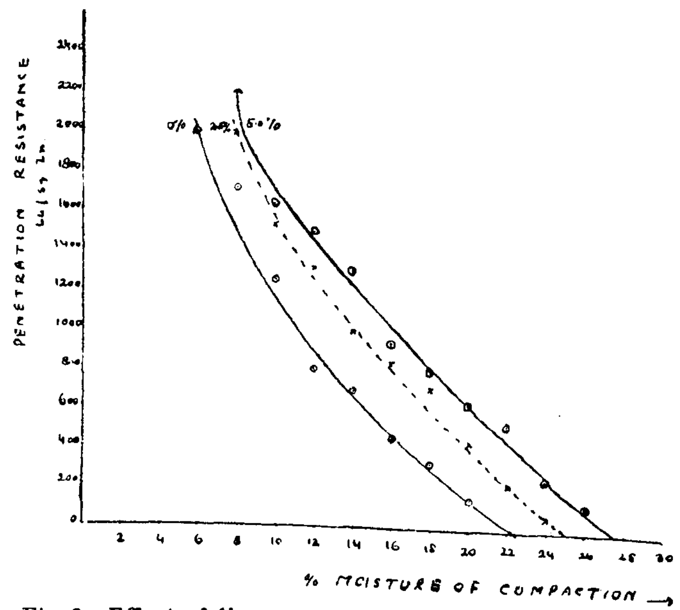


Fig. 2. Effect of lime on the penetration resistance of black cotton soil

TABLE 4

Penetration Resistance (lb per sq. in.) of compacted lime-soil mass at different moisture content (Needle area 1/20 sq. inch)

Percentage moisture of compaction	Blank	2.5 per cent lime	5.0 per cent lime	7.5 per cent lime	10.0 per cent lime
6.0	2000 lb (Proctor's needle cannot measure above 2000 lb)				
8.0	1720 lb	2000 lb	Above 2000 lb	Above 2000 lb	Above 2000 lb
10.0	1260	1540	1640	1680	1680
12.0	800	1320	1500	1680	1680
14.0	700	1000	1300	1400	1560
16.0	460	840	920	1100	1200
18.0	340	720	800	1000	1000
20.0	160	440	640	680	880
22.0	Too soft	240	520	440	520
24.0	,,	80	260	340	400
26.0	,,	Too soft	120	100	140

Saturation moisture is about 40 per cent

TABLE 5

Absorption and penetration resistance of lime-soil blocks kept over wet sand for 48 hrs

Percentage lime	Soil No. 2		Soil No. 6		Soil No. 8	
	Percentage absorption	Penetration resistance lb per sq. in.	Percentage absorption	Penetration resistance lb per sq. in.	Percentage absorption	Penetration resistance lb per sq. in.
Blank	—	Softened	47.66	Swelling & softening	43.33	Swelling & softening
5.0	38.27	Softened	34.97	440 lb	33.86	400 lbs
7.5	31.20	310 lbs	—	—	—	—
10.0	29.62	500	32.11	660	25.13	600
15.0	25.47	500	32.18	800	27.18	680
20.0	—	—	32.67	1020	31.87	1080
25.0	—	—	32.90	1180	30.75	1160

TABLE 6

Penetration resistance of lime-soil blocks kept over wet sand for a period of one month.
Soil No. 2. P. I. = 35. Sand = 10 per cent

Per cent lime		Penetration resistance lb per sq. in.
5		500
10		900
15		1400
20		2000
25	above	2000

It will appear from Table 4 that the adition of lime to black cotton soil increases its resistance to penetration at different moisture contents. The appreciable increase in the penetration resistance of lime-soil compacted mass, although influenced by moisture and density, has been largely increased by the presence of lime, especially when the results are considered at higher moisture contents.

(ii) Resistance to penetration after allowing lime to set

Lime-soil blocks with varying percentages of lime weighing 200 gms., compacted at optimum moisture were taken for the study. These blocks after having been cured for a week were placed over sand filled in small moulds having perforated base. These moulds containing soil blocks were kept over wet sand with sufficient water floating in a tray and then absorption as well as resistance to penetration with Proctor's needle studied. Results obtained are given in Table 5.

It will appear from Table 5 that blocks containing 5 per cent or more of lime retained enough stability under wet conditions as indicated by the resistance to penetration of Proctor's needle. Apart from this, capillary absorption is also reduced with lime.

It has also been observed that owing to slow chemical action of lime, prolonged contact of lime-soil compacted blocks with capillary moisture considerably increases their strength even in wet state. This is shown by the increase in penetration resistance of lime-soil block (Soil No. 2) kept over wet sand for a period of one month. Results are given in Table 6.

In the above experiments, the lime-soil blocks used were cured for a period of one week in humid air. Another set of

similar experiment was carried out with lime-soil blocks which have been cured for a period of one month and two months separately. Results obtained are given in Tables 7 & 8.

TABLE 7

Capillary absorption of lime-soil blocks curing period = 1 month

Soil No.	Lime per cent	Penetration resistance lb per sq. in.	Absorption in 48 hrs. per cent
3	5	200	37.51
	10	520	37.57
	15	720	35.83
	20	1000	41.55
	25	1600	39.51
5	5	240	44.33
	10	600	37.30
	15	1200	34.07
	20	1520	34.76
	25	1600	37.16

TABLE 8

Soil No.	Lime per cent	Penetration resistance lb per sq. in.	Absorption in 48 hrs. per cent
3	5	280	31.28
	10	1900	32.85
	15	2000	27.68
	20	2080	28.88
	25	Needle did not penetrate. Leaves only a mark.	—
5	5	Block broken.	—
	10	760	29.13
	15	Needle penetrated only to ½ in. at 105 reading of proctor needle.	26.98
	20	Needle did not penetrate. Leaves only a mark.	26.60
	25	do.	29.16

It will appear from Tables 7 and 8 that an appreciable increase in the strength of lime-soil blocks can be easily attained by giving longer period of curing and this fact points to the suitability of lime for improving black cotton soil in road construction.

EFFECT OF SAND

Use of granular material like sand (moorum could also possibly be used) has also been tried in increasing proportions for stabilizing black cotton soil. This was done to find out how far this process compares with lime treatment. Effect of increasing percentage of sand on the shrinkage of black cotton soil is given in Table 9 and reproduced in Fig. 3.

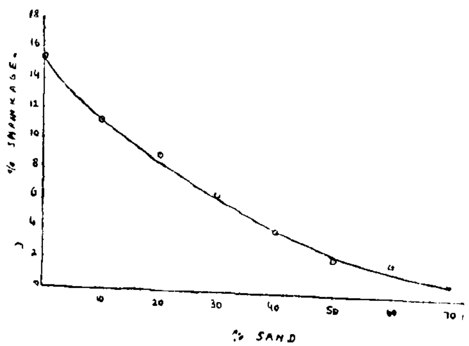


Fig. 3. Effect of increasing percentage of sand on the shrinkage of black cotton soil

TABLE 9

Sand percentage	Percentage B. C. soil	Density	Volumetric shrinkage Percentage
0	100	1.979	15.53
10	90	1.906	11.43
20	80	1.917	9.03
30	70	1.928	6.41
40	60	1.905	4.18
50	50	1.866	2.40
60	40	1.869	2.26
70	30	1.813	0.91

Effect of adding sand on moisture absorption and softening characteristics of black cotton soil as indicated by resistance to penetration with Proctor's needle was studied. Results obtained are given in Table 10.

TABLE 10

Absorption of water by compacted black cotton soil blocks mixed with increasing percentage of sand

Percentage black cotton soil	Percentage sand	Absorption after 1 hour Per cent	Remarks
80	20	37.27	Swelling & softening of blocks observed.
70	30	36.91	Complete softening and less of swelling.
60	40	34.14	Complete softening.
50	50	34.15	do.
40	60	31.34	do.
30	70	27.96	do.

It will appear from Tables 9 and 10 that addition of sand causes reduction in shrinkage but does not prevent its softening under worse moisture conditions. These experiments clearly indicate that addition of large proportion of sand no doubt cuts down shrinkage and swelling of black cotton soil, but does not improve its bearing power under worse moisture conditions. On the other hand addition of lime has a distinct advantage over sand in this respect.

EFFECT OF LIME + SAND

With a view to cut down the concentration of lime for stabilizing purpose, its use in combination with increasing percentage of sand has also been tried.

Effect of 2.5 per cent and 5.0 per cent lime mixed with varying proportion of sand on (i) shrinkage (ii) water resistance and (iii) resistance to penetration with Proctor's needle was studied. Results obtained are given in Tables 11 and 12. The blocks were cured for a week before testing.

TABLE 11

The effect of lime+sand on shrinkage and water resistance property of black cotton soil

S. No.	Description	Vol. Shrinkage percentage	Density	Time of disintegration
6	2.5 per cent lime+20 per cent sand	5.17	1.784	3 Minutes
	,, ,, +30 ,, ,,	3.81	1.792	4 ,,
	,, ,, +40 ,, ,,	2.40	1.794	20 ,,
	,, ,, +50 ,, ,,	1.38	1.795	34½ ,,
	5 per cent lime +20 ,, ,,	3.0	1.750	9 ,,
	,, ,, +30 ,, ,,	2.86	1.766	5 months
	,, ,, +40 ,, ,,	1.97	1.763	5 months
	,, ,, +50 ,, ,,	1.30	1.768	9 months
7	2.5 per cent lime+20 ,, ,,	8.41	1.754	3 Minutes
	,, ,, +30 ,, ,,	5.85	1.762	3 ,,
	,, ,, +40 ,, ,,	3.77	1.725	2½ ,,
	,, ,, +50 ,, ,,	2.11	1.734	3½ ,,
	5 per cent lime +20 ,, ,,	6.25	1.711	3 ,,
	,, ,, +30 ,, ,,	4.57	1.699	3 ,,
	,, ,, +40 ,, ,,	2.70	1.705	3 ,,
	,, ,, +50 ,, ,,	2.02	1.717	7 ,,

TABLE 12

Soil No.	Description	Penetration resistance lb per sq. in.	Absorption Percentage
6	2.5 per cent lime+20 per cent sand	40	38.34
	,, ,, +30 ,, ,,	56	35.85
	,, ,, +40 ,, ,,	88	30.72
	,, ,, +50 ,, ,,	100	30.29
	5 per cent lime +20 ,, ,,	300	29.48
	,, ,, +30 ,, ,,	350	30.88
	,, ,, +40 ,, ,,	500	26.71
	,, ,, +50 ,, ,,	650	21.19
7	2.5 per cent lime+20 ,, ,,	48	30.43
	,, ,, +30 ,, ,,	56	33.38
	,, ,, +40 ,, ,,	92	40.49
	,, ,, +50 ,, ,,	210	40.06
	5 per cent lime +20 ,, ,,	106	40.58
	,, ,, +30 ,, ,,	142	29.47
	,, ,, +40 ,, ,,	164	33.44
	,, ,, +50 ,, ,,	250	33.89

It will appear from Tables 11 and 12, that the addition of lime + sand cuts down shrinkage besides improving water-resistance property and resistance to penetration under worse moisture conditions.

COMPRESSIVE STRENGTH OF LIME-SOIL BLOCKS

Lime-soil blocks 2.5 in. dia. and 1½ in. high prepared at optimum moisture and cured for a week in humid air with subsequent drying were tested for compressive strength. Results obtained are given in Table 13.

TABLE 13

Compressive strength of lime-soil (black-cotton) mixture. Soil No. 3

Percentage lime	Density	Compressive strength Initial (lb per sq. inch)	Compressive strength Final (lb per sq. inch)
0	2.230	503	604
5.0	1.807	338	434.5
7.5	1.857	246.0	349.0
10.0	1.738	202.0	233.0
15.0	1.630	207.0	251.0
20.0	1.590	157.0	224.0
25.0	1.530	269.0	380.8

EFFECT OF CEMENT

Effect of increasing quantities of cement on the shrinkage of different black cotton soils was tried and results obtained are given in Table 14 and reproduced in Fig. 4.

It will appear from Table 14 that addition of cement has more or less the same effect as lime in reducing shrinkage of lime.

In order to explore the possibility of using cement for cement-soil roads in black cotton soil areas, the resistance of cement-soil blocks compacted at optimum moisture to wetting and drying cycles was also studied. Results obtained are given in Table 15.

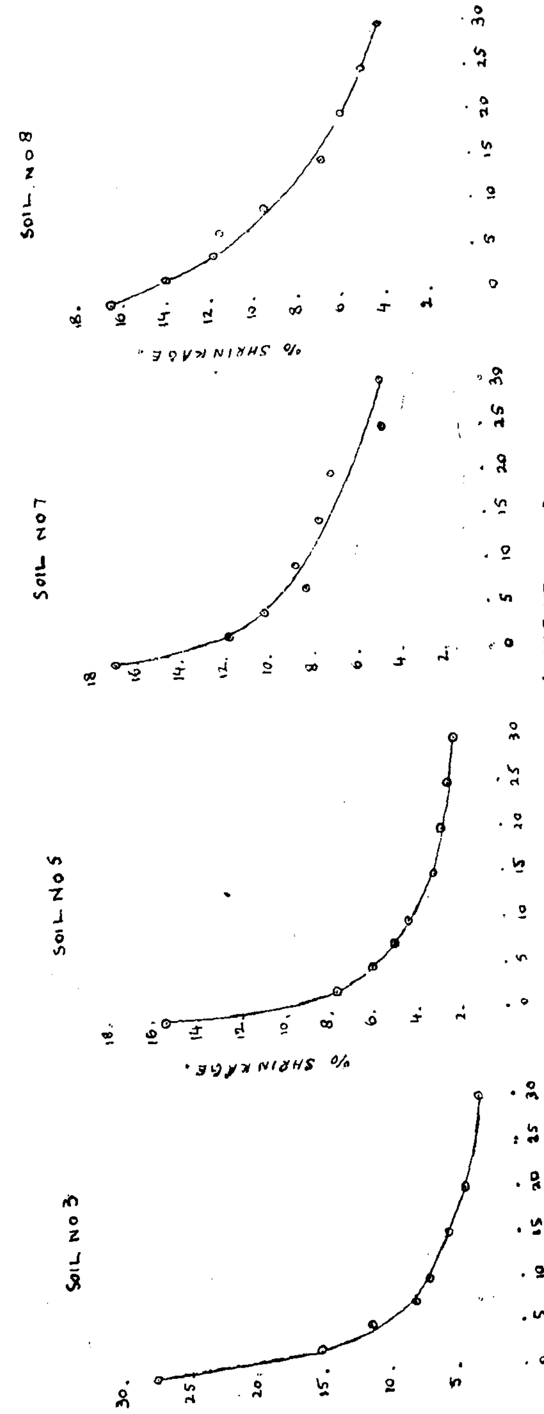


Fig. 4. Effect of cement on the volumetric shrinkage of black cotton soils

TABLE 14

Effect of cement on the shrinkage of black cotton soil

Percentage cement	Soil No. 3		Soil No. 4		Soil No. 5		Soil No. 6		Soil No. 7		Soil No. 8	
	Density	Shrinkage	Density	Shrinkage	Density	Shrinkage	Density	Shrinkage	Density	Shrinkage	Density	Shrinkage
0	2.23	27.76	1.887	14.38	—	15.52	1.801	11.89	1.798	17.03	1.758	16.62
2.5	1.97	15.37	1.835	9.66	1.868	7.73	1.760	8.18	1.726	11.82	1.860	14.09
5.0	1.87	11.47	1.834	7.24	1.882	6.01	1.805	7.41	1.724	10.26	1.859	11.86
7.5	1.80	8.05	1.836	7.00	1.858	5.06	1.778	6.81	1.711	8.27	1.882	11.59
10.0	1.81	6.99	1.841	6.09	1.870	4.32	1.814	5.96	1.731	8.78	1.903	9.44
15.0	1.81	5.36	1.813	5.57	1.844	3.18	1.791	4.04	1.732	7.55	1.835	6.76
20.0	1.80	4.02	1.820	4.80	1.836	2.62	1.784	3.44	1.734	6.92	1.849	5.82
25.0	—	—	1.809	4.44	1.824	2.33	1.779	3.25	1.726	4.57	1.822	4.85
30.0	1.75	2.93	1.806	2.90	1.822	1.95	1.800	2.76	1.722	4.55	1.818	3.98

TABLE 15

Loss in weight of cement-soil (black cotton) mixture through a process of wetting and drying (12 cycles)

Soil No. 3													
Cement percentage	Density	1st cycle	2nd cycle	3rd cycle	4th cycle	5th cycle	6th cycle	7th cycle	8th cycle	9th cycle	10th cycle	11th cycle	12th cycle
5.0	1.87	Disintegrated in 10 minutes											
7.5	1.80	11.11	82.40	Complete disintegration									
10.0	1.81	1.43	16.43	29.99	57.30	85.87	Complete disintegration						
15.0	1.81	+6.1	4.43	19.01	22.44	32.63	35.80	45.48	52.29	57.38	65.38	72.16	77.35
20.0	1.80	+6.0	+6.30	+3.14	+2.65	+0.97	2.89	5.23	9.88	17.49	19.20	24.58	25.64
30.0	1.75	+6.5	+8.05	+6.67	+7.51	+7.66	+6.72	+6.72	+6.87	+4.28	+2.62	1.28	0.8
40.0	1.77	+6.70	+8.16	+7.40	+9.27	+10.05	+9.72	+10.42	+11.32	+10.82	+12.55	+9.55	+10.57
50.0	1.76	+5.01	+7.24	+6.32	+8.03	+8.77	+8.63	+9.22	+10.35	+9.71	+12.78	+12.72	+8.85

It will appear from Table 15 that a fairly high percentage of cement is needed to improve the weather resisting quality of black cotton soil. Besides high cost, even the uniform mixing of cement with black cotton soil under field conditions may not prove to be an economic possibility.

EFFECT OF LIME + CEMENT

Since the addition of lime makes the black cotton soil friable and reduces its plasticity, it was thought advisable to try varying concentration of cement and lime mixed together. Effect of varying concentrations of lime + cement on the shrinkage and compressive strength was studied. Results obtained are given in Table 16.

TABLE 16

Effect of lime + cement on the characteristics of black cotton soil

Lime per cent	Cement per cent	Density	Shrinkage per cent	Compressive strength	
				Initial crack lb per sq. in.	Final crack lb per sq. in.
0	0	2.23	27.76	503	604
5	5	1.756	8.10	940.80	1093.10
5	7.5	1.713	7.75	1388.8	1469.40
5	10.0	1.716	7.36	1394.2	1478.0
5	15.0	1.733	5.53	1581.0	1733.70
5	20.0	1.703	4.10	2150.0	2150.00
0	0	2.23	27.76	503	604
7.5	5.0	1.776	8.81	1120.0	1394.20
7.5	7.5	1.732	6.91	1120.0	1388.0
7.5	10.0	1.717	6.50	985.6	1097.6
7.5	15.0	1.712	5.14	1394.2	1612.8
7.5	20.0	1.702	3.51	—	—
0	0	2.23	27.76	503.0	604.0
10	5	1.56	6.79	627.0	716.8
10	7.5	1.69	6.27	896.0	940.8
10	10.0	1.57	5.76	806.0	1061.7
10	15.0	1.60	4.34	985.6	1276.8
10	20.0	1.57	3.26	716.8	1254.4

It will appear from Table 16, that the addition of lime and cement in combination does not show marked effect on the

TABLE 17

Loss in weight of lime + cement-soil (black cotton) mixture through a process of wetting and drying (12 cycles)

Soil No. 3													
Cement percentage	Lime percentage	1st cycle	2nd cycle	3rd cycle	4th cycle	5th cycle	6th cycle	7th cycle	8th cycle	9th cycle	10th cycle	11th cycle	12th cycle
5.0	5.0	1.17	47.10	100.0	—	—	—	—	—	—	—	—	—
7.5	5.0	0.39	8.61	47.32	100.0	—	—	—	—	—	—	—	—
10.0	5.0	+1.94	3.41	11.16	100.0	—	—	—	—	—	—	—	—
15.0	5.0	+4.10	+6.65	+2.79	18.28	52.24	52.74	66.45	67.27	68.48	76.15	77.32	83.08
20.0	5.0	+1.97	+6.19	4.06	3.70	5.42	4.05	5.91	8.37	12.38	12.23	16.50	19.65
5.0	7.5	+0.79	0.60	5.23	24.61	68.97	70.88	100.0	—	—	—	—	—
7.5	7.5	+2.28	+3.23	0.14	3.45	6.41	12.49	54.97	100.0	—	—	—	—
10.0	7.5	+2.88	+4.89	+3.32	0.61	6.60	9.26	11.98	15.26	18.09	21.11	100.0	—
15.0	7.5	+3.92	+5.83	+5.53	+4.36	+4.54	+3.89	+1.54	4.65	11.06	12.69	13.59	16.93
20.0	7.5	2.18	5.66	+5.22	+5.13	+6.42	+5.07	+4.36	+3.48	+2.74	+1.25	+1.75	0.11
5.0	10.0	+5.68	+6.81	+3.49	8.63	100.0	—	—	—	—	—	—	—
7.5	10.0	+2.18	+3.82	+2.11	3.08	73.93	76.25	78.06	100.0	—	—	—	—
10.0	10.0	+6.41	+8.99	+8.49	+7.98	+7.34	+6.37	+3.63	+0.71	64.92	67.95	100.0	—
15.0	10.0	+7.28	+9.62	+9.80	+9.74	+10.16	+11.0	+9.38	+8.81	+8.34	+6.50	+3.65	+1.10
20.0	10.0	+7.31	+10.40	+10.40	+11.18	+11.77	+11.96	+11.72	+11.77	+12.30	+11.58	+12.30	+11.23

shrinkage compared to when each of the admixture is used separately. The main advantage of using lime + cement together is that the presence of cement considerably improves its compressive strength. Presence of lime, however, will ensure easy and uniform mixing with cement.

Resistance of (lime + cement-soil) compacted blocks to the effect of wetting and drying cycles was also studied. Results obtained are given in Table 17.

It will appear from Table 17 that 15–20 per cent cement + 10 - 7.5 per cent lime are needed to improve the weather resistance quality of black cotton soil. Use of such high concentration of admixtures may prove to be uneconomical in practice.

EFFECT OF LIME + PLASTER OF PARIS AND CEMENT + PLASTER OF PARIS

With an object to cut down shrinkage of black cotton soil drastically, addition of lime + plaster of paris (burnt gypsum) and cement + plaster of paris has also been tried. Results obtained are given in Tables 18 and 19.

TABLE 18

Effect of lime + plaster of Paris on the characteristics of black cotton soil

Soil No. 2

Percentage lime	Percentage plaster of Paris	Density	Percentage shrinkage	Time of disintegration
0	0	—	29.12	—
2.5	1.0	1.96	18.01	2 minutes
do.	2.5	1.96	17.34	4 ,,
do.	5.0	1.86	10.76	4 ,,
5.0	1.0	1.80	10.20	4 ,,
do.	2.5	1.77	7.69	6 ,,
do.	5.0	1.70	6.74	7 ,,
7.5	1.0	1.73	7.07	35 ,,
do.	2.5	1.68	4.29	42 ,,
do.	5.0	1.67	1.70	84 ,,
				percentage loss (after 5 hrs. immersion)
10.0	1.0	1.70	6.78	47.38 %
do.	2.5	1.67	4.35	50.0 %
do.	5.0	1.62	1.29	71.03 %

TABLE 19

Effect of cement + plaster of Paris on the shrinkage of black cotton soil

Soil No. 2

Percentage cement	Percentage plaster of Paris	Density	Percentage shrinkage
0	0	—	29.12
5.0	1.0	1.89	11.44
do.	2.5	1.89	8.31
do.	5.0	1.85	5.31
7.5	1.0	1.91	8.87
do.	2.5	1.87	6.76
do.	5.0	1.86	5.15
10.0	1.0	1.88	5.61
do.	2.5	1.88	5.57
do.	5.0	1.88	4.67
15.0	1.0	1.86	4.79
do.	2.5	1.86	4.77
do.	5.0	1.86	4.11

It will appear from Tables 18 and 19 that addition of plaster of Paris in combination with lime or cement appreciably cuts down shrinkage.

But before recommending the use of gypsum mixed with lime or cement for stabilizing black cotton soil, its resistance to weathering cycles and compressive strength should be taken into account.

CONCLUSION

It has been shown in the above study that use of admixtures such as lime, cement and gypsum etc., in small concentrations can be tried for improving physical characteristics of black cotton soil. Among all these admixtures, use of hydrated lime appears to give quite encouraging results. Use of 10 per cent lime, which can be further reduced if there is any possibility of economically bringing sand and mixing with the soil before treatment, can be recommended for stabilizing black cotton soil in the base. The base thus treated will considerably help in reducing the pavement thickness in black cotton soil areas, besides improving the service value of road constructed over a lime stabilized base.

ACKNOWLEDGMENT

Shri R. N. Kapur, Research Assistant has assisted the Authors in the investigation of above study.

REFERENCES

1. Road construction in black cotton soil by Rao Bahadur A. Nageswara Ayyar (I. R. C. Journal Vol. X-1, Dec. 1945).
2. The upward movement of water and salt solution in the black cotton soil by L. A. Ramdas and A. K. Malick. (Proceedings of the Indian Academy of Science, Vol. XVI, July 1942).
3. The effect of chloride on the black cotton soil previously treated with sodium carbonate by P. K. Katti. (Proceedings of the Indian Academy of Science, Vol. XXIX, 1944).
4. Use of stabilized soil in engineering construction, Section IV, by Shri S. R. Mehra & Shri H. L. Uppal. (I. R. C. Journal, Vol. XV-2, Nov. 1950).

"STABILISATION OF LATERITIC SOILS"

By

E. K. RAMACHANDRAN*, B.E., M. TECH.

CONTENTS

1. SCOPE OF THE INVESTIGATION

This investigation attempts to study the engineering properties of the local (Kharagpur) lateritic soil. It is a low-level lateritic soil which has got a fairly good distribution all over the Midnapur district. Various special methods have been suggested in dealing with laterite soils. The purpose of this investigation is to verify the applicability of these methods to the Kharagpur soil.

The local soil contains a high percentage of fines. In order to bring down the percentage of fines to the desired value addition of locally available cheap materials like river sand (from Kosai River, Midnapur) and top soil (Kharagpur) has been tried and their influence on the engineering properties of the resulting soil-mixture studied in the laboratory.

Another object of the investigation is to study the susceptibility of the local soil to stabilisation and to throw some light on the relative efficiency of various admixtures. (Portland cement, $CaCl_2$, Shelmac R.C. 3 etc.).

* Indian Institute of Technology, Kharagpur.

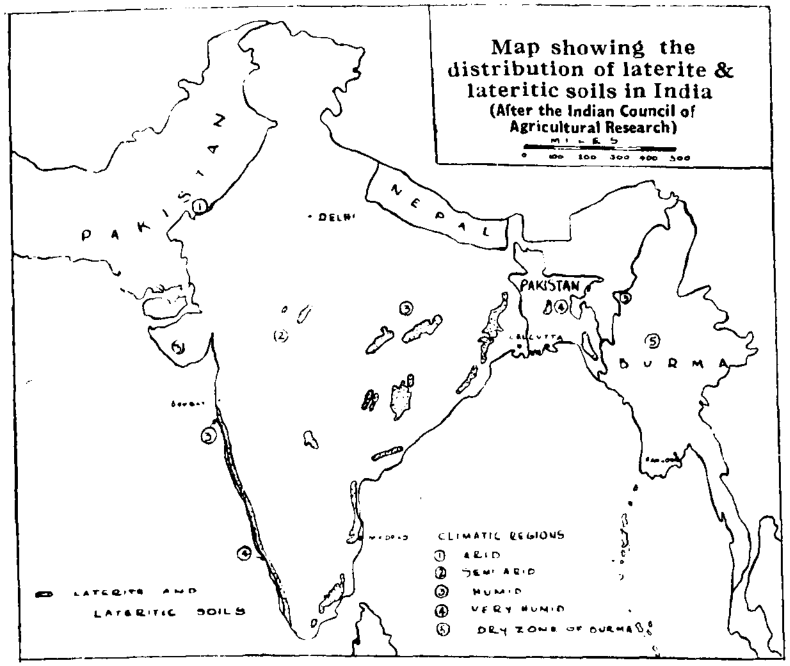


Fig. 1.

An attempt has also been made to assess the utility value of the local soil for road construction as surface and base courses.

2. SOIL COMBINATIONS

Though slight variations occur from place to place, in general, there are two distinct types of soils in the locality which are normally available within reasonable depth. They are :

(i) A top soil usually available within 2 ft to 3 ft of the surface ;

(ii) A coarser soil called the lateritic soil under the above layer.

(iii) In addition, local river sand available within a lead of 5-6 mile has also been used in this investigation (Kosai River, Midnapore). The particle size distribution of each of the above soils has been been given in Fig. 2. Various proportions of the local sand and top soil have been added to the lateritic soil in an

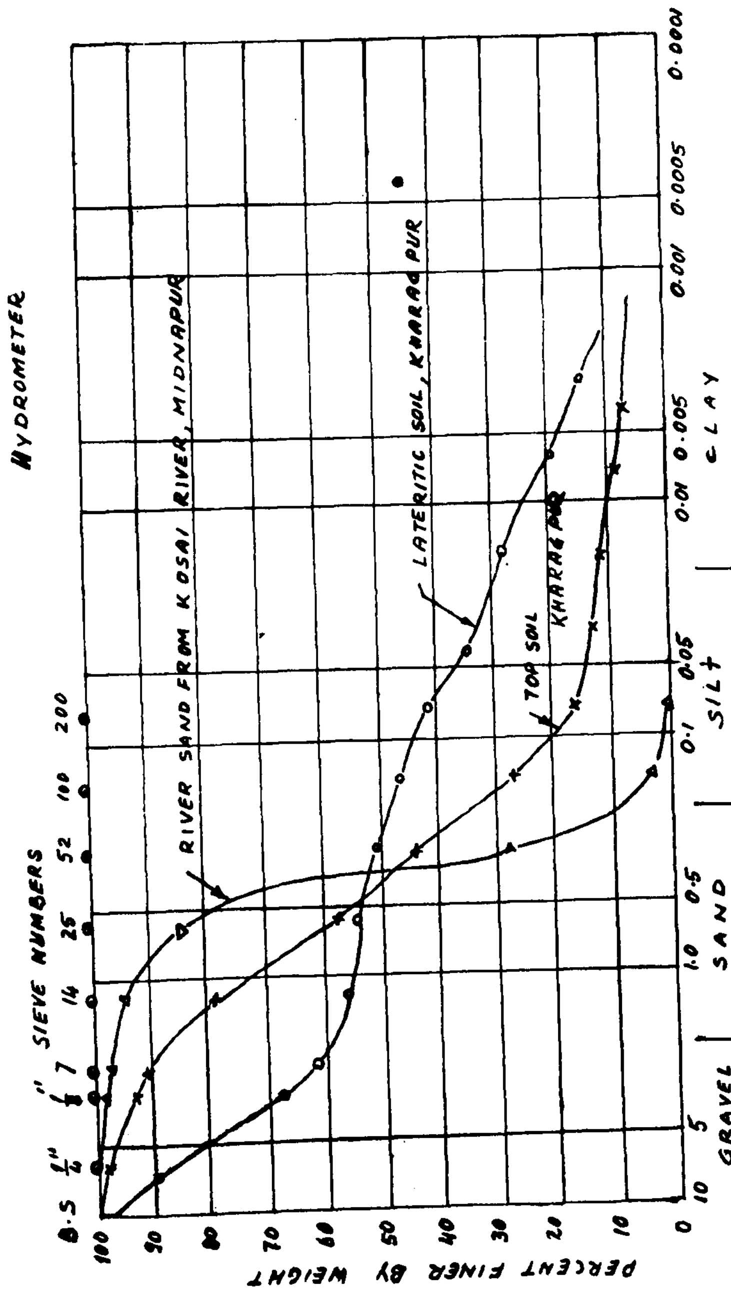


Fig. 2. Grain size distribution curves of local soils available for stabilization

attempt to study whether any improvement could be effected on the engineering properties of the resulting soil mixtures.

The soil samples were collected from pits within the Institute premises (5 ft × 5 ft size and about 8 ft deep). The samples were pulverised, care being taken not to crush the individual particles and then allowed to dry at the room temparature. The field moisture contents of the samples were also determined.

It was decided to utilise both these (sand and top soil) for stabilising lateritic soil. In addition, various percentages of cement, calcium chloride and Bitumen (Shelmac RC_3) were also employed as stabilisers in this investigation.

3. TESTS CARRIED OUT

Various tests were carried out to determine the engineering properties of the soil and soil mixtures. The behaviour of soil cement mixture was studied in greater detail in view of the availability of Portland cement and its known behaviour and properties. Similarly sand-soil mixtures were also studied in some detail.

The tests conducted include:

(1) Grading and particle distribution.

(2) Atterberg limits.

(3) Specific gravity.

(4) Compaction characteristics.

(5) Unconfined compressive strength of specimens having a depth : diameter ratio of 2 (8 in. × 4 in. dia. specimens).

(6) California bearing ratio test and

(7) Direct shear test.

A brief description of the tests together with the results obtained and inferences drawn is given below:

(1) Grading and particle size distribution

Tests on grading and mechanical analysis were carried out on the representative samples of soils collected and the results are shown in Fig. 2.

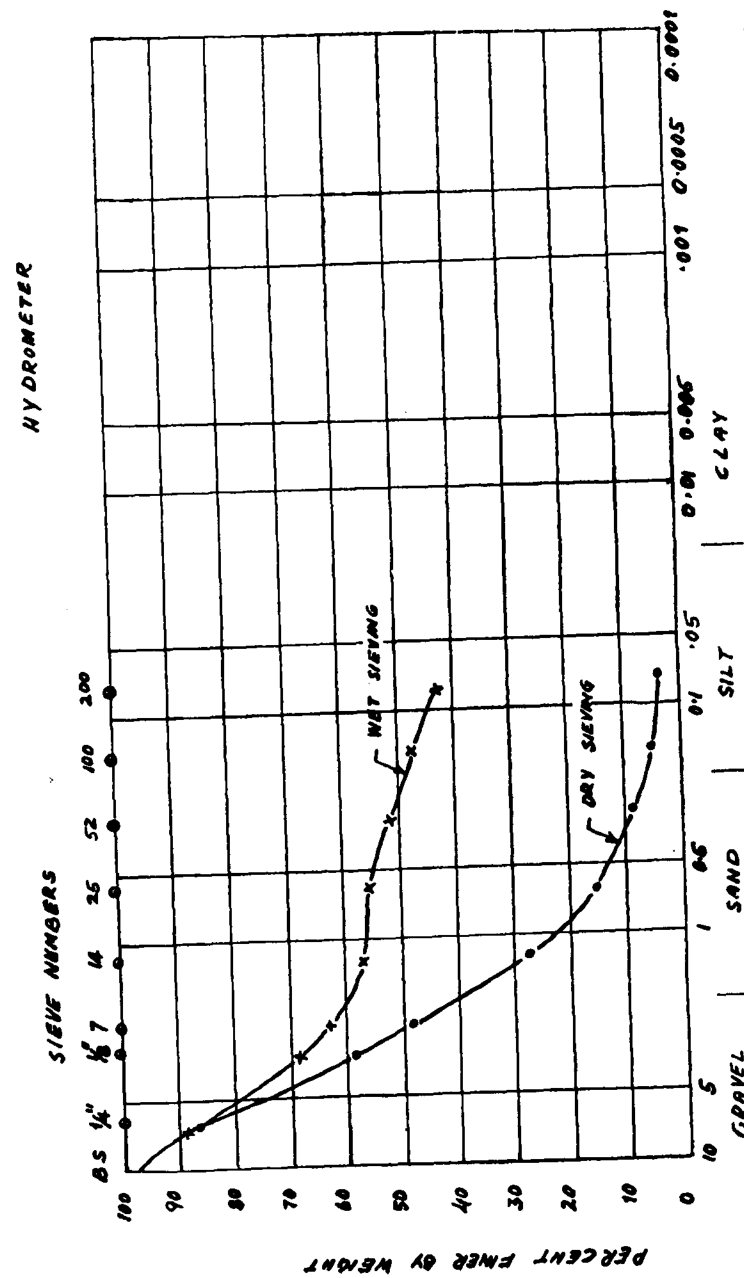


Fig. 2·A. Sieve analysis: Wet sieving and dry sieving
Soil: Lateritic soil from Kharagpur

Previous investigations have emphasised the necessity for wet sieving in the case of lateritic soil in order to get a correct idea of the grain size distribution. The nodules contain good amount of fines which can be separated only by wet sieving. The two Sieve Analysis curves, one for wet sieving and the other for dry sieving, have been shown in Fig. 2-A for a comparative idea.

The particle size Analysis (hydrometer analysis) was carried out for the lateritic soil after dispersion of the soil suspension with hydrogen peroxide. The test was also conducted without any deflocculating agent to study the effectiveness of the H_2O_2 as a deflocculating agent. The two curves have been shown in Fig. 2-B. It can be seen that for correct idea of the particle size distribution, full deflocculation has to be effected by the use of some suitable agents. The ineffectiveness of the usual deflocculating agents in the case of lateritic soils has been repeatedly emphasised by various writers [1], [2].

The particle size distribution curves, Fig. 2, show that:

(1) The top soil has about 70 per cent of it lying between 1 mm. and 0.1 mm. and about 6–7 per cent of clay.

(2) The lateritic soil is fairly well graded and has only 10 per cent of it lying between 1 mm. and 0.1 mm. The clay content is about 12 per cent.

(3) The river sand is poorly graded and has about 90 per-cent of it between 0.1 and 1.0 mm.

Type of Soil	Clay .002 mm per cent	Silt .002–.02 mm per cent	Fine sand .02–.2 mm per cent	Coarse sand .02–2 mm per cent	Gravel 2 mm per cent
Lateritic soil	11.0	18.5	20.0	11.5	39
Top soil	6.0	5.5	22.0	55.5	11.0
Sand	—	—	8.0	89.0	3.0

With a view to assess the suitability of the individual soils for stabilisation purposes, the limits of the grading curves for soils best suited for stabilisation as recommended by Hogentogler and by the Public Roads Administration Classification

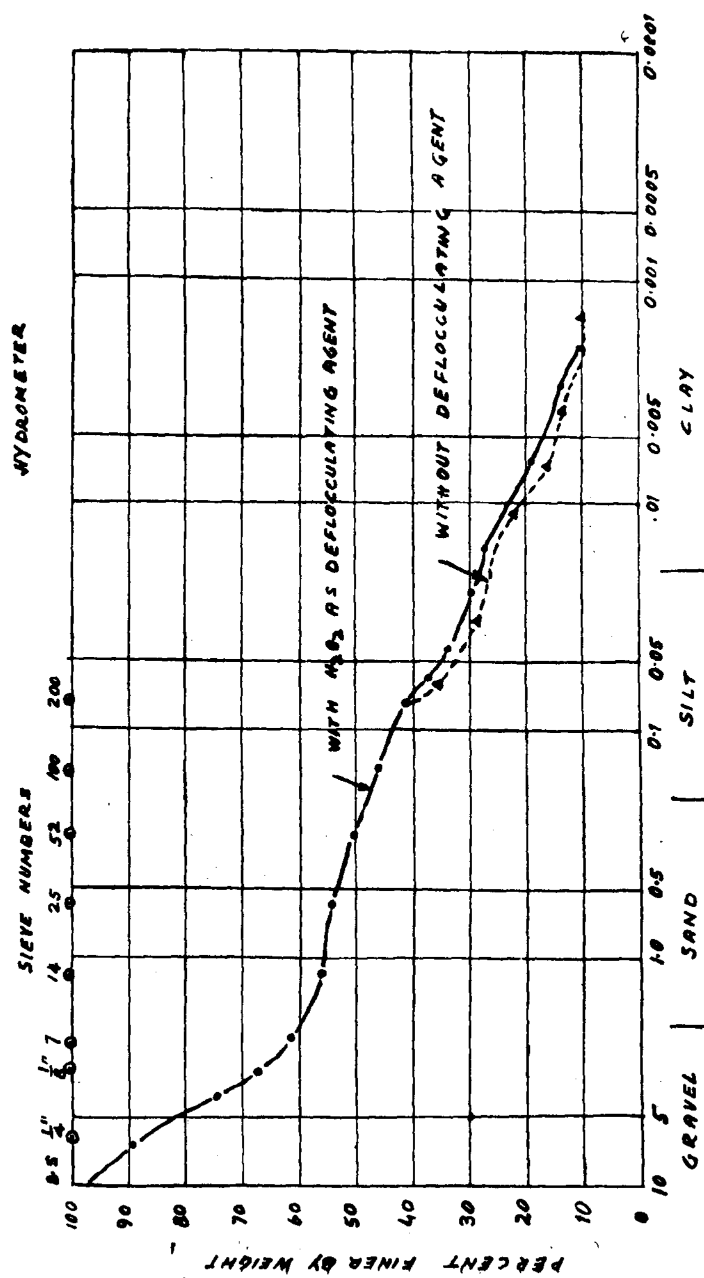


Fig. 2-B. Hydrometer analysis with and without deflocculating agent

have been reproduced in Figs. 20 and 21. The grading curves of the soil and soil mixtures have been plotted in the same graph for comparative study.

The figures indicate that the lateritic soil contains a high percentage of fines and in order to bring down this, addition of about 30 per cent sand is required acccording to the P. R. A. curves. The limiting curves by Hogentogler indicate that addition of 50 per cent sand is necessary to keep the soil curve within the limiting values. Since Hogentogler's curves essentially deal with the finer fractions of the soil only, addition of 30 per cent of sand as indicated by the P. R. A. curves may be taken to give a grading best suited for stabilisation for all practical purposes. Addition of greater percentages of sand is not advisable from the point of view of cost.

(2) Atterberg limits

The Atterberg limits of top soil and lateritic soil were found out using the standard apparatus (on—36 B.S.) and the results are tabulated below :

Soil Type	Liquid limit	Plastic limit	Placticity index	Shrinkage limit
Lateritic	(i) 51.5	(i) 16.45	(i) 35	(i) 9.4
Soil	(ii) 46.0	(ii) 14.19	(ii) 32	(ii) —
Local Top	(i) —	(i) 10.6	(i) 22	
Soil	(ii) 33	(ii) 10.8	(ii) 22	—

Remillon[2] has expressed the opinion that the Atterberg limits of lateritic soils as obtained by the usual procedure are deceptive because the mixing of particles during the determination of liquid limit leads, on the one hand, to a breaking up of fines and, on the other hand, to a slight deflocculation. These deflocculated fines absorb an additional amount of water. The limits obtained by experimental results depend on the modification the material has undergone in the course of the determination of the liquid limit.

From the above results the laterite soil may be classified under A6 according to the P.R.A. Classification.

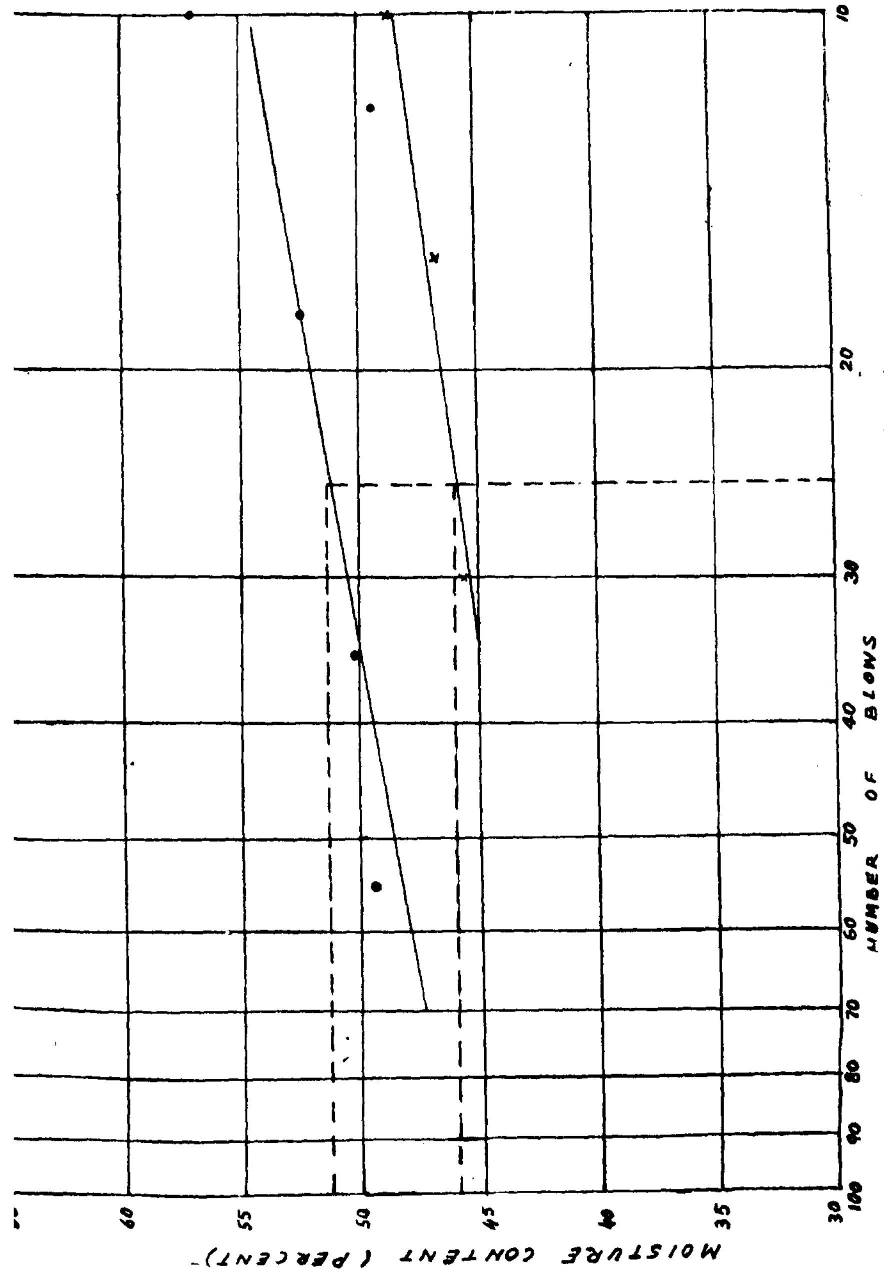


Fig. 3. Liquid limit of lateritic soil from Kharagpur

(3) Specific gravity

The specific gravities of the solid soil particles of lateritic soil, top soil and sand have been found and tabulated below :

Tye of Soil	Specific Gravity
Lateritic soil	2.70
Top soil	2.56
Sand	2.64

(4) Compaction characteristics

Tests were carried out to determine the maximum dry density and optimum moisture content for the top soil, lateritic soil and soil mixtures. These were done according to A.S.T.M. standards with the Proctor mould using 5½ lb hammer and a 12 in. drop. Figs. 4, 5, 6, 7, & 8 show the compaction curves giving dry density in lb per cu. ft. against moisture content percentage for the two soils and soil mixtures—5 per cent and 10 per cent of top soil and 5 per cent, 10 per cent, 15 per cent and 20 per cent of sand and 2½ per cent and 5½ per cent of $CaCl_2$ by dry weight of

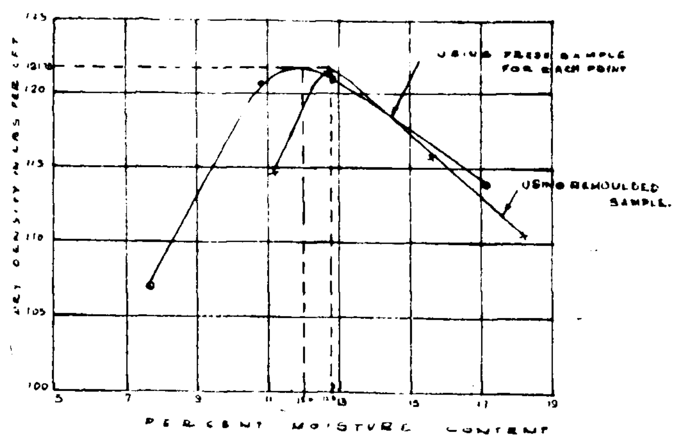


Fig. 4. Effect of remoulding on dry density and moisture content

Soil : Lateritic soil from Kharagpur.
Compaction : Std. Proctor.

lateritic soil were employed as admixtures and their compaction characteristics studied. Fresh samples were used for each point on the Proctor curve.

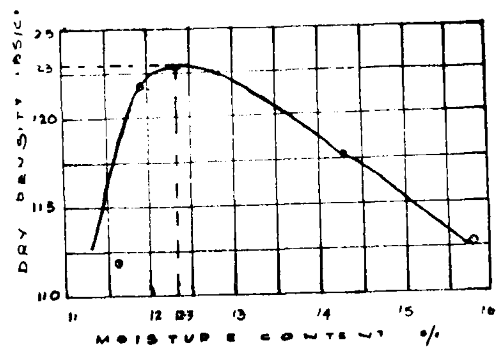


Fig. 5. Compaction test on top soil (Kharagpur)
N.B. : **Soil collected from I.I.T. college campus**

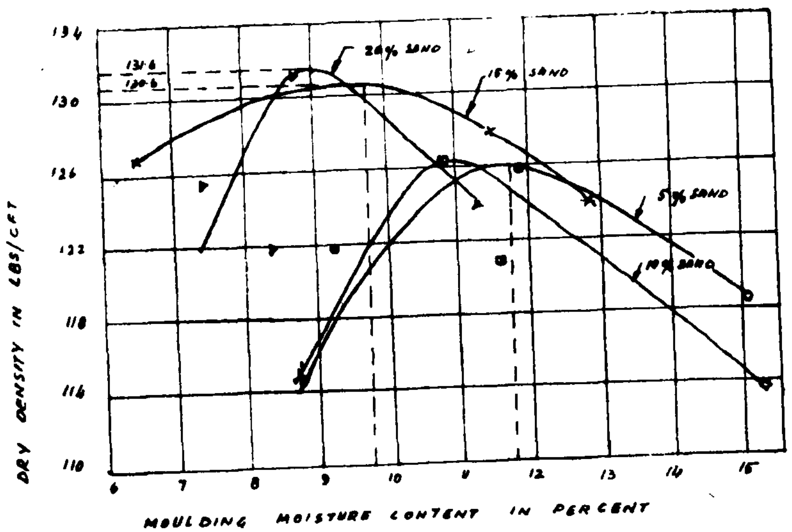


Fig. 6. Compaction characteristic effect of addition of sand to lateritic soil

Soil : Lateritic soil from Kharagpur
Sand : Kosai river, Midnapur

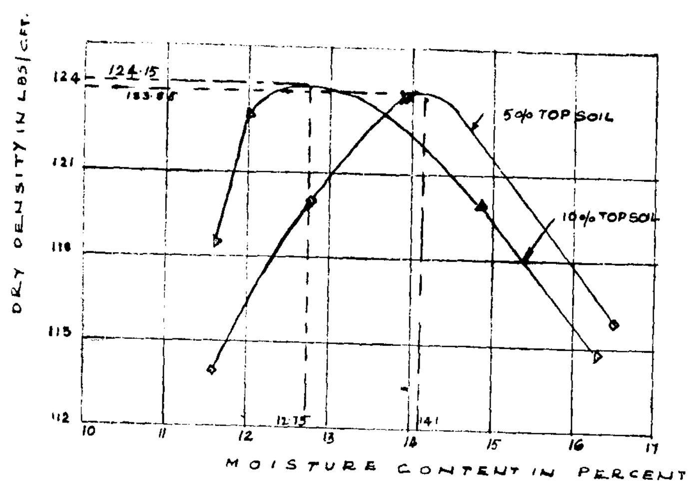


Fig. 7. **Compaction characteristic effect of addition of top soil to lateritic soil**

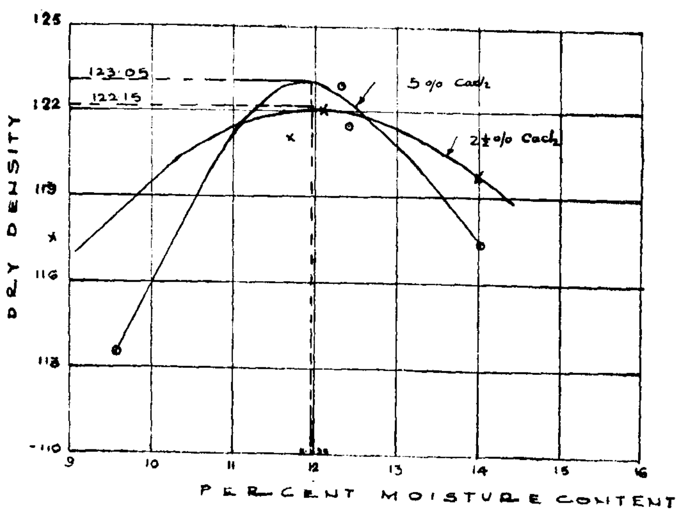


Fig. 8. **Compaction characteristic effect of addition of $CaCl_2$ to lateritic soil**

The effect of varying compactive efforts on the dry density of lateritic soil was also investigated. Fig. 10 gives the moisture content dry density relationships for various compactive efforts.

Fig. 9 shows that a straight line relationship exists between compactive effort and dry density when plotted on a semilog scale.[2]

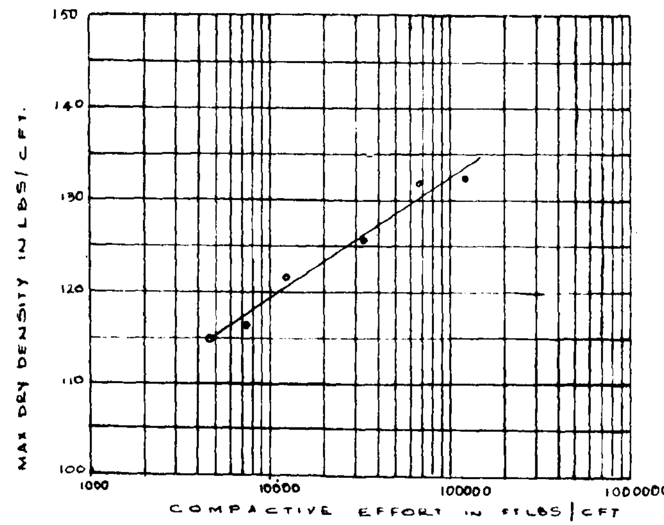


Fig. 9. **Variation of maximum dry density as a function of the work of compaction**

The need for using fresh samples for each point on the Proctor curve has been emphasised by Remillon[2]. To study the effect of remoulding on density of the local lateritic soil, 2 sets of Proctor tests were conducted, one set using the same sample for all the points on the Proctor curve and the other using fresh samples each time. Fig. 4 shows the moisture content—dry density relationships. It may be inferred that the effect of repeated use of the same sample on density is not considerable on the local soil.

The following Table gives the optimum Proctor density and

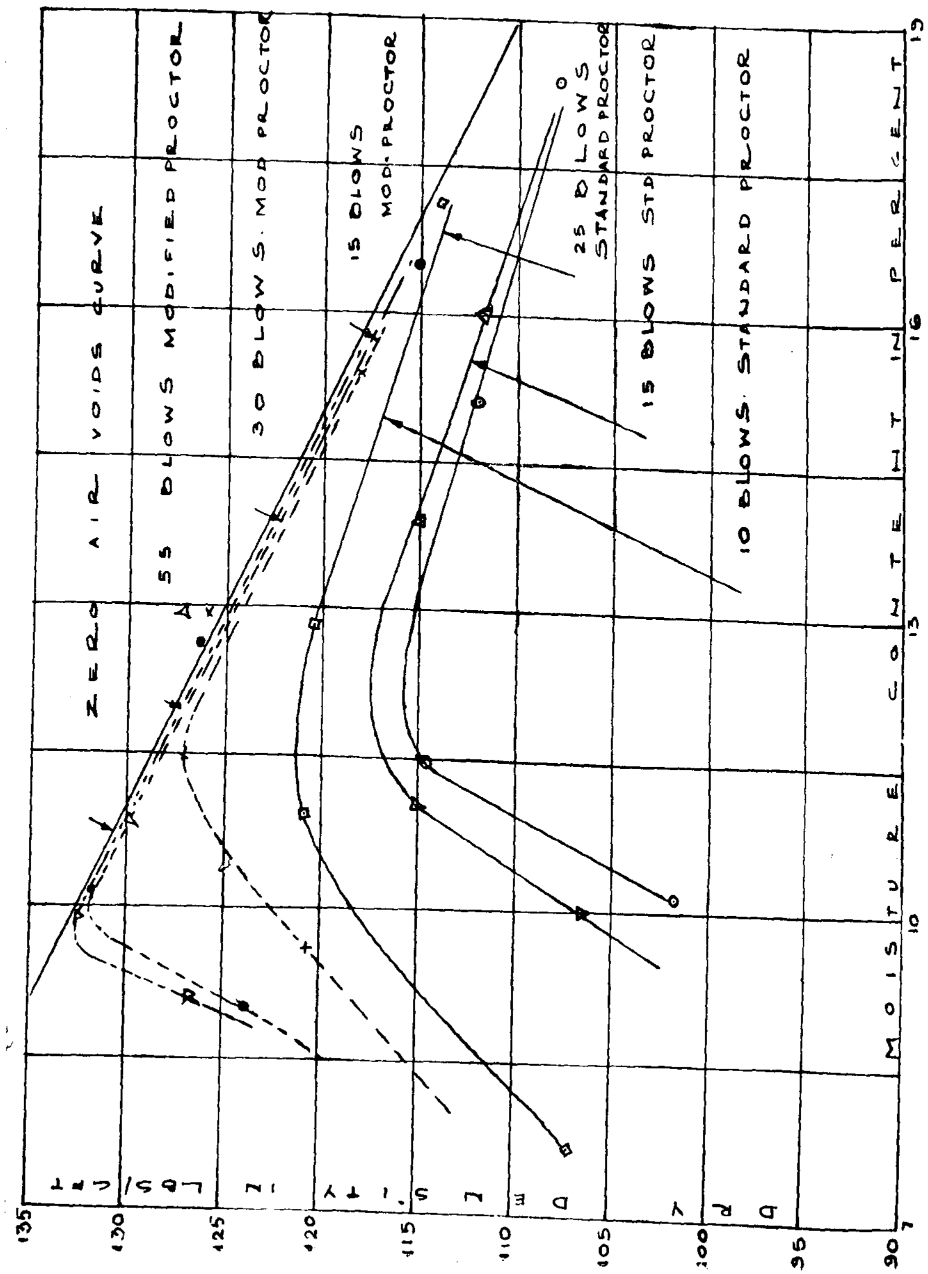


Fig. 10. Effect of compactive effort on dry density

Soil: Lateritic soil

optimum moisture content for the above two soils and soil mixtures :

Soil type	Max. dry density (lb per cu. ft.)	Optimum moisture content (percentage by dry wt. of soil)
Lateritic soil	121.75	12.00
Top soil	123.00	12.35
Lat. soil + 5 per cent top soil	123.85	14.10
Lat. soil + 10 per cent top soil	124.15	12.75
Lat. soil + 5 per cent sand	126.00	11.75
Lat. soil + 10 per cent sand	126.40	11.00
Lat. soil + 15 per cent sand	130.60	9.75
Lat. soil + 20 per cent sand	131.60	9.0
Lat. soil + $2\frac{1}{2}$ per cent $Cacl_2$	122.00	12.00
Lat. soil + 5 per cent $Cacl_2$	123.00	11.80

The results of the tests show that :

(1) The maximum dry density is increased and the optimum moisture content is decreased with increasing percentage of admixtures like sand and top soil. (Figs. 5, 6, 7, 8).

(2) The optimum moisture contents in general, for the varying combinations, fall within 9 to 14 per cent, the lower value for 20 per cent sand and the higher for 5 per cent top soil.

(5) Unconfined compressive strength

In order to study the susceptibility of the lateritic soil to stabilisation and also the relative efficiency of various admixtures used, cylindrical specimens of soil and soil mixtures were prepared and tested after 7 days of curing. Two specimens were prepared in each case and one was air dried for 7 days while the other was air dried for one day and moist cured under wet sand for 6 days before testing for compressive strength.

The specimens were prepared in a split brass mould 4 in. in diameter and 8 in. high. In each case, the respective optimum moisture content as obtained from Proctor compaction tests was used in the preparation of the specimen. The specimens were prepared in three layers using the same compactive effort per unit volume as for Proctor *i.e.*, 12400 ft lb per cu. ft. The C.B.R. apparatus was used as a compression machine and the ultimate compressive strengths of the cylinders were found out.

In the case of cement, 12 per cent of water which is the optimum moisture content for lateritic soil alone was added for all the specimens. In the case of Shelmac RC_3 the fluidity of the admixture was taken into account. The following Table gives the compressive strength of 4 in. × 8 in. stabilised blocks (wet and dry cured):

Description of admixture	Compressive strength in p.s.i.		Nature of failure
	Air cured for 7 days	Moist cured for 7 days	
Lateritic soil only	(1) 147	—	Crumbly
	(2) 101.5		Shear
Lat. soil+5 per cent top soil	(1) 163.5	—	
	(2) 148.0	—	Shear
Lat. soil+10 per cent top soil	(1) 151.5	—	Shear
	(2) 134.0	—	
Lat. soil+5 per cent sand	(1) 143.5	—	do.
	(2) 119.0		
Lat. soil+10 per cent sand	(1) 152.5	—	Crumbled
	(2) 172.0	—	Swelled and cracked
Lat. soil+15 per cent sand	(1) 215.0		
	(2) 171.5	—	Shear
Lat. soil+20 per cent sand	(1) 173.0		
	(2) 177.0	—	80 Cracked
Lat. soil+2½ per cent cement	139.5	Crumbled before test	
Lat. soil+5 per cent cement	140.0	23.7	Shear
Lat. soil+7½ per cent cement	231.0	127.5	,,
Lat. soil+10 per cent cement	319.0	177.0	,,
Lat. soil+15 per cent cement	291.0	139.5	,,
Lat. soil+2½ per cent Shelmac RC_3	14.35	—	Plastic
Lat. soil+5 per cent Shelmac RC_3	18.30	—	Plastic cum-Shear
Lat. soil+7½ per cent Shelmac RC_3	11.90	—	Plastic
Lat. Soil+10 percent Shelmac RC_3	7.95	—	,,
Lat. Soil+2½ per cent $CaCl_2$	4.00	Broke into two before test	,,
Lat. Soil+5 per cent $CaCl_2$	28.7	4.0	,,

The results show that:

(1) Cement and sand give very good compressive strengths which increase with the increase in the percentage of admixture added.

(2) $CaCl_2$ and Shelmac RC_3 give very poor results.

(3) 10 per cent of top soil and 10 per cent sand gave almost the same value of unconfined compressive strength.

(4) The optimum amount of sand is found to be 15 per cent. Further addition of sand reduced the unconfined compressive strength, Fig. 11.

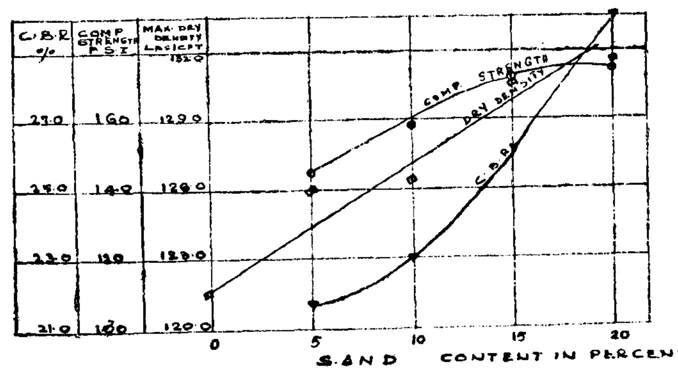


Fig. 11. **Properties of soil-sand mixtures correlated to sand content**

Soil : Lateritic soil, Kharagpur
Sand : Kosai river sand (6 miles from Kharagpur)

(5) The optimum cement content is found to be 7½ per cent which gave a value of 231 p.s.i. Higher percentages of cement are not necessary. The British standard specifications lay down a strength of 250 lb per sq. in. as the strength sufficient for stabilisation of a soil for highway purposes. Therefore 7½ per cent of cement by weight of dry lateritic soil may be considered sufficient for stabilisation purposes. This inference is further confirmed by the C.B.R. values

obtained for $7\frac{1}{2}$ per cent cement-soil mixture, Fig. 12.

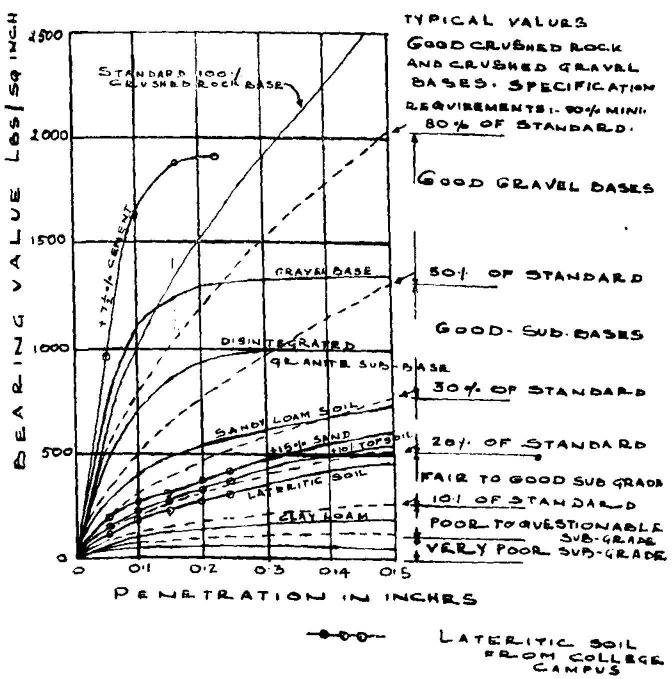


Fig. 12. **Load penetration curves of typical soils tested by C.B.R. method**

(Taken from Highway Engg. by Hewes & Oglesby)

(6) It may be inferred from the above results and also Fig. 14 that addition of bitumen above 5 per cent is not necessary.

To ascertain the effect of addition of bitumen on the cohesive strength of soil mixtures, a number of triaxial shear tests should be done before drawing any conclusions. Graphs have been shown in Figs. 11, 13 & 14 connecting cement content and sand content and bitumen content to unconfined compressive strength.

(6) California Bearing Ratio Test

C.B.R. Test was conducted on lateritic soil and soil mixtures. In preparing the samples C.B.R. optimum moisture

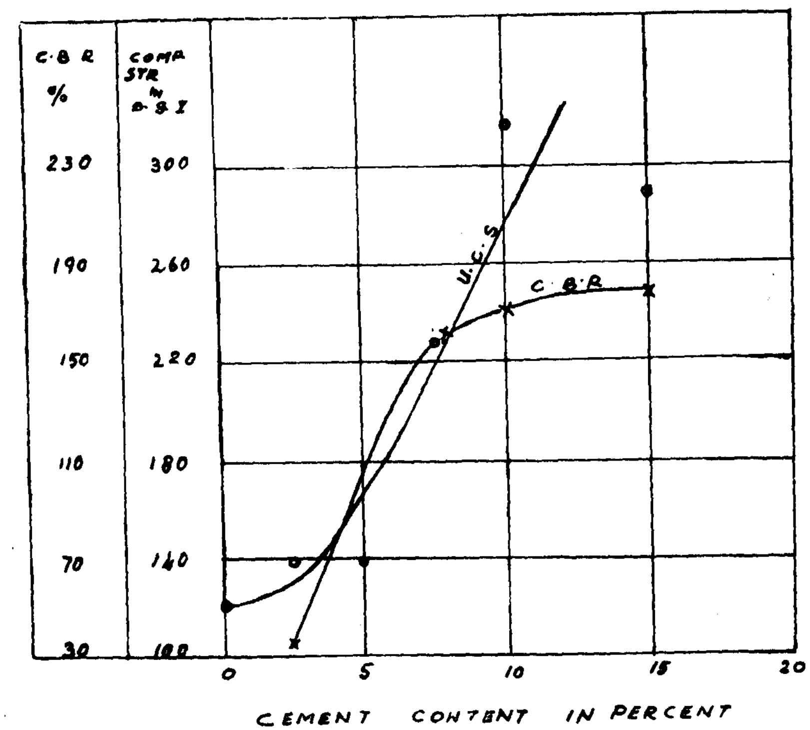


Fig. 13. **Effect of cement content on C.B.R. and unconfined compressive strength**

Soil: Lateritic soil from Kharagpur

content was assumed to be 2 per cent less than the corresponding Proctor optimum moisture content.

C.B.R. specimens were prepared for varying moisture contents and the C.B.R. values were found before and after 4 days soaking in water. The results are tabulated below: (Also see Fig. 15)

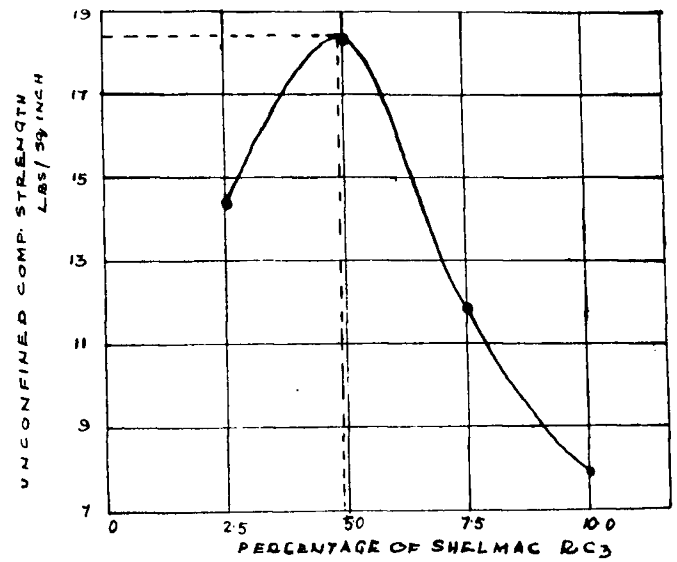


Fig. 14. **Effect of bitumen content on unconfined compressive strength**

Soil: Lateritic soil from Kharagpur

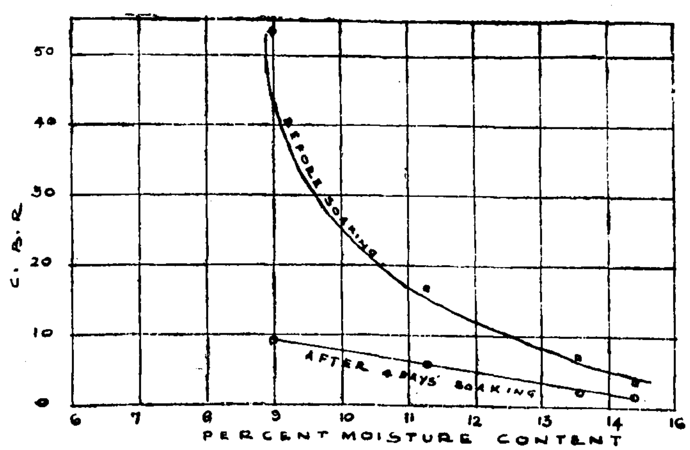


Fig. 15. **Effect of moulding moisture content on C.B.R.**

Soil: Laterite soil from Kharagpur

C.B.R. of lateritic soil

Moisture content per cent	C.B.R. before soaking per cent	C.B.R. after soaking per cent
14.4	3.7	1.8
9.0	53.3	9.3
19.2	1.70	...
13.3	7.3	2.3
11.3	17.0	6.0

Values of C.B.R. were found for soil mixtures also and they have been shown below:

C.B.R. for soil mixtures

Description	C.B.R. before soaking
Lateritic soil + 5 per cent top soil	24.3
do. + 10 per cent top soil	20.3
do. + 5 per cent sand	21.7
do. + 10 per cent sand	23.0
do. + 15 per cent sand	26.2
do. + 20 per cent sand	30.0

The values of C.B.R. for soil-cement mixtures after 7 days of moist curing were also found out with a view to correlate them with the corresponding values of compressive strength of 4 inch × 8 inch soil-cement cylinders. They have been tabulated below:

Description	C.B.R. after 7 days moist curing
Lateritic soil + 2½ per cent cement	37
do. + 5 per cent cement	72.4
do. + 7½ per cent cement	165.4
do. + 10 per cent cement	170.0
do. + 15 percent cement	176.0

Fig. 16 shows the relation between C.B.R. and unconfined compressive strength for lateritic soil-cement mixtures. The graph shows that a linear relation exists between C.B.R. and unconfined compressive strengths for the soil cement mixtures[5].

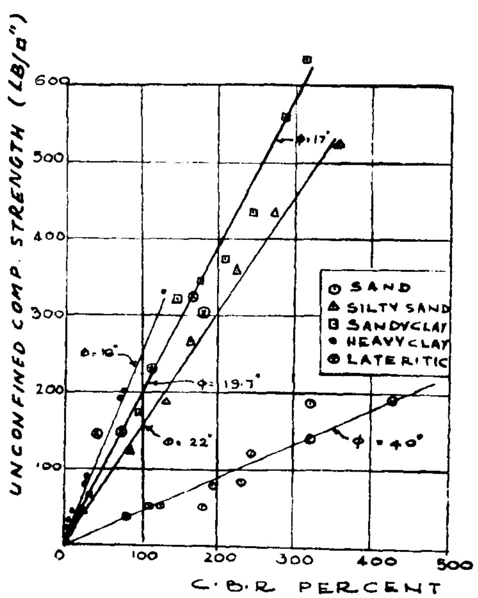


Fig. 16. **Relation between C.B.R. and unconfined compressive strength for soil-cement mixtures**

(After Mr. D. J. Maclean, Road Research Lab., D.S.I.R., U.K.)
N.B.—Soil: Kharagpur lateritic soil

From the graph, it may be inferred that even though the local soil contains a good percentage of coarser fractions, it behaves as a sand-clay mixture, (in fact both the curves coincide) when stabilised with cement.

It has been suggested[6] that a laboratory value of 120 per cent of C.B.R. and field value of 80 per cent of C.B.R. would be sufficient for soil-cement mixtures. Figs. 12 and 13 would indicate that 5 per cent to 7½ per cent of cement is quite sufficient for good road performance and values above this are uneconomical and unnecessary. Addition of sand increases the C.B.R. value in direct proportion to the sand content. The results coupled with those obtained for unconfined compressive strength

tests indicate that 15 per cent of sand gives fairly good value for purposes of highway design.

Figs. 11 and 13 show the ralationship of cement content and sand content to C.B.R.

It may be seen from Fig. 12 that the lateritic soil without any admixture will serve as a fair to good subgrade with the usual compaction. Addition of 10 per cent of top soil and 15 per cent of sand to this soil increases the C.B.R. value to 20-30 per cent which falls within the region of "very good subgrade". 7½ per-cent of cement gives a very high value of C.B.R. equal to that of good crushed rock and crushed gravel bases.

(7) Direct shear test (Controlled strain type)

A limited number of quick-quick tests[6] were conducted on loose and stabilised soils and the results are tabulated below:

Description	ϕ in degrees	'C' in lb per sq.in.
Sand loose	29.67	—
Lat. soil loose	19.70	2.04
Lat. soil stabilised	39.40	11.33
Lat. soil + 10 per cent sand	32.30	10.70
Lat. soil + 15 per cent sand	42.00	7.10

Specimens were prepared to give the maximum dry density using the Proctor optimum moisture content. Judging from the limited number of tests conducted, the addition of sand upto 10 per cent does not seem to have any appreciable effect on the values of C & ϕ. A number of tests will have to be performed before arriving at any definite conclusion. The triaxial compression test may be preferable to the direct shear test in that, in the preparation of laboratory specimens as well as testing, field conditions could be approached better in the case of the former.

Fig. 17 [2]shows the stress envelope corresponding to the effect of a 5½ ton wheel load on the road surface. For a satisfactory strength of the road bed, it is sufficient if the intrinsic curve of the material does not cut the stress envelope. Based on this, Remillon[2] suggests that it is sufficient if the cohesion is at least equal to 7.1 p.s.i., since the majority of laterites have angles of friction of at least 40 deg. Therefore, before constructing paved highways on laterites one must make certain that at least this amount of cohesion is obtained.

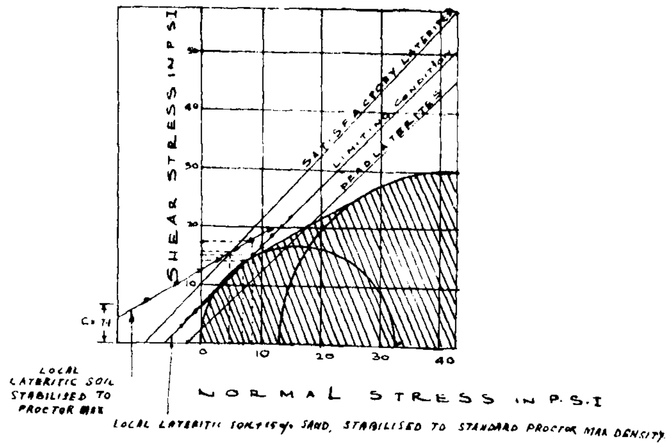


Fig. 17. **Stress envelope for 5-ton load**

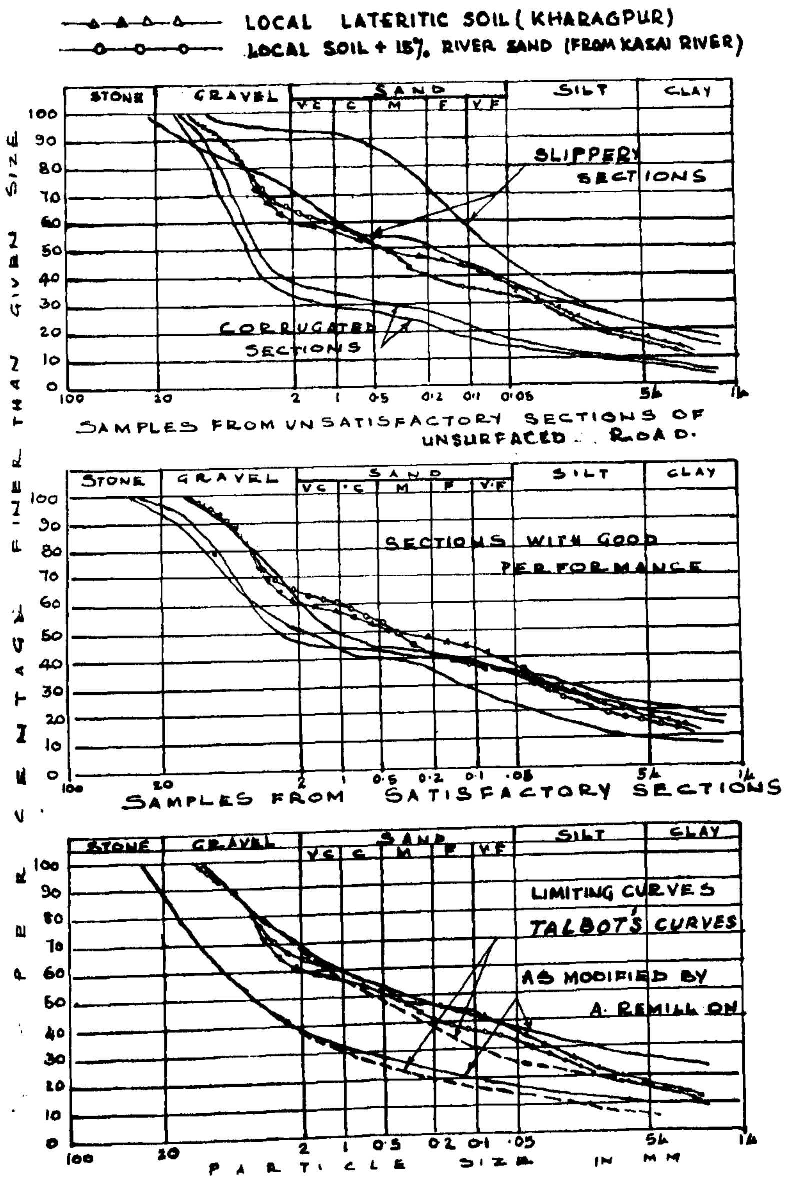
(Ref: 2, H.R.B. Bulletin 108)

Local soil stabilised as per standard Proctor gives a value of $\phi = 39.4$ deg. and $C = 11.33$ p. s. i. Addition of 15 per cent of sand considerably increases the angles of internal friction (42 deg.) whereas the reduced cohesion value is just within the prescribed minimum (7.1 p. s. i.)

4. UTILISATION OF LATERITES FOR THE CONSTRUCTION OF UNPAVED ROADS

Remillon[3] has given typical curves based on his tests made on sections that during the dry season formed corrugations and also on sections that during the rainy season either showed a lack of stability or excessive slipperiness, Fig. 18. The curve envelopes of materials that give satisfactory performance have been derived from a number of granulometric analyses and have also been reported.

In the small size range these envelopes differ from the formerly accepted curves of Talbot[4]. The medium coarse



Limiting curves for lateritic soils for good performance as surface course

Fig. 18. **Granulometric analyses: unsurfaced roads**

(After A. Remillon, France, Ref: H.R.B. Bulletin 108)

content must exceed the Talbot value if the road surface is to retain a good riding quality. It has also been suggested by Remillon that the clay content should lie between 10 & 25 per cent. The curves for the local soil and soil mixture (15 per cent sand) have been inserted to study the utility value of the soil. Both the curves lie within the proposed limits suggested by Remillon. But the plasticity indices of satisfactory materials should lie between 10 and 25 per cent. The local soil does not satisfy the plasticity condition. Therefore, addition of 15 per cent of sand is necessary and might bring down the plasticity index within the required limits.

It is quite likely that a detailed investigation into the plasticity characteristics of the soil mixtures would throw some more light on the utility value of the local lateritic soil for surface courses.

However, it may be safely concluded that the local soil with 15 per cent of river sand offers an ideal soil mixture highly suitable for the construction of unpaved roads. The values of C.B.R., maximum dry density and unconfined compressive strength for the above soil mixture also confirm the above view.

5. UTILITY OF LOCAL SOIL FOR PAVED HIGHWAYS

The grading limits for stabilisation fixed by the U.S. Public Roads Administration and the limiting curves proposed by Hogentogler for good soil mortars have been reproduced in Figs. 20 and 21. It has also been found that in order to keep the grading curve of the local soil within the above limits addition of 30-50 per cent of sand is necessary. Evidently this resulting mixture will have a plasticity index far below that of the original soil.

From Fig. 12, it can be seen that the soil on stabilisation by compaction only can serve as a good subgrade. Addition of sand improves the resistance to penetration and also shear strength of the soil as indicated by the Figure.

Cement stabilisation (5-7½ per cent) is also found to give very good base course comparable to those of crushed gravel and rock, Fig. 12.

In the light of the values suggested by Remillon[a] for ϕ and C^{a}(40 deg. & 7.1 p. s. i.) the local soil with 15 per cent of sand might prove to be a good material for road base.

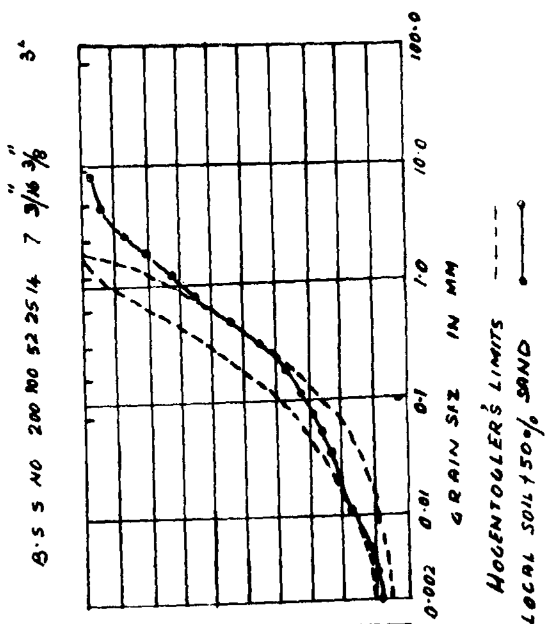


Fig. 20. Grading of good soil mortars (Hogentogler)

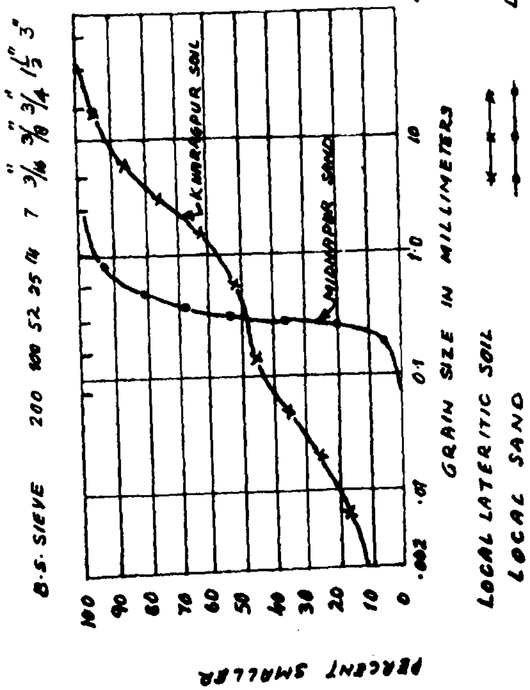


Fig. 19. Mechanical analysis of local soils available for stabilization

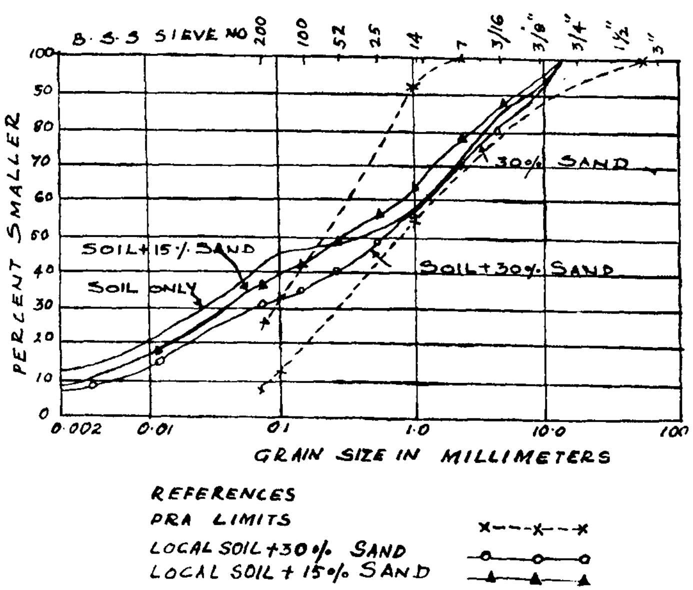


Fig. 21. **Grading limits for stabilization fixed by the U.S. Public Roads Administration system**

Soil : Kharagpur lateritic soil
Sand : Kosai river sand, Midnapur

The strength attained by soil-sand mixture by restraint in the lateral direction should be taken care of. This is neglected in the unconfined compressive strength test but is accounted for in the triaxial test. Hence the need for doing a number of triaxial tests before assessing the shear strength of the stabilised soil and its utility for road construction.

6. CONCLUSIONS

It should be stated that the results of this investigation may be affected somewhat by factors which could not be evaluated because of limited time. Among those variables are the variation in fineness of samples and possible variations in shape and composition of particles in different samples. With these limitations the following conclusions may be drawn from the data given in this note.

(A) Mechanical properties

(1) As different from other lateritic soils, the nodules are strong and not porous.

(2) The fine particles are less flocculated. It is difficult to give exact values of Atterberg limits for reasons already explained. For two consecutive determinations, the values of liquid limit gave a difference of 5 per cent.

(3) As per Remillon[2], all granulometric curves of laterites show a discontinuity between 2 and 0.5 mm. showing that nodules do not form in a continuous manner from the fines.

The granulometric curve for the local soil shows discontinuity between 1.2 mm. and 0.3 mm., Fig. 2. In addition, the soil contains about 40 per cent of -200 materials and a good percentage of fine sand.

(4) It is better to use fresh sample for each point on the Proctor curve while compaction tests are conducted.

(5) Wet sieving is necessary to get a clear idea of particle size distribution.

(B) Classification

The local soils possess the following characteristics :

Clay	Silt	Fine sand	Coarse sand	Gravel
11 per cent	18.5 per cent	20 per cent	11.5 per cent	39 per cent

Liquid limit between 51.5 & 46.0

Plasticity index 32–35

Shrinkage limit 9.4

Specific gravity 2.70 (This high value indicates a high percentage of iron)

The soil may be classified as belonging to A-6 (P. R. A.)

(C) Compaction Characteristics

(1) The maximum dry density obtained by Proctor compaction only is 121.5 lb per cu. ft.

(2) The addition of admixtures like sand and soil increases the maximum dry density and decreases the optimum moisture content with increasing precentage of admixture.

(3) The variation of maximum dry density can be expressed as a function of work of compaction (straight line relation) when plotted on a semi-log scale.

(D) Unconfined compressive strength and C. B. R. and shear tests

(1) The soil itself without any admixtures develop a strength of 100 p.s.i. and above for Proctor compaction. (12400 lb per cu. ft.)

(2) Addition of sand upto 15 per cent increases the unconfined compressive strength; afterwards the unconfined compressive strength decreases. This is due to the fact that the strength due to the lateral restraint is neglected in the test. Therefore, triaxial shear tests are necessary.

(3) Optimum cement percentage is 5–$7\frac{1}{2}$ per cent as can be seen from Fig. 12 and also Fig. 13.

(4) Bitumen (Shelmac RC_3) content over and above 5 per cent does not add to the strength and should be avoided.

(5) The C.B.R. increases with sand content and cement content.

(6) The loose soil has a value of about 19.7 deg. and $C = 2$ p.s.i.

The stabilised soil has a value of 39 deg. and $C = 11.33$ p. s. i. Addition of sand reduces cohesive value and increases the angle of internal friction. From these considerations, 15 per cent of sand may be considered to be the optimum value (Fig. 17).

(E) Utility for road construction

(1) Figs. 20 and 21 indicate that addition of 30 per cent to 50 per cent of sand is necessary to make the soil suitable for stabilisation as per Hogentogler and P.R.A. curves.

(2) The local soil with 15 per cent sand is highly suited for unpaved roads, Fig. 18.

(3) The lateritic soil behaves exactly as a sand clay mixture as regards its compressive strength and C.B.R., when mixed with cement, Fig. 16, irrespective of the coarser fractions it contains. The straight line relationship also may be noted.[5]

(4) The soil can be best stabilised by any other soil containing a high percentage of particles between 1.2 mm and 0.3 mm and a low percentage of clay.

(5) Soil stabilised with 5—$7\frac{1}{2}$ per cent Portland cement can serve as a very good base course.

In conclusion the results of the experiments indicate that the local lateritic soil can be well graded by the addition of materials (like Kosai River sand) having a high percentage of particles between 1.2 mm and 0.3 mm. The possibility of stabilising such soil-sand mixtures with economical quantities of cement deserves proper attention.

REFERENCES

(1) Lateritic Soils and their Stabilisation by Hans F. Winterkorn and E. C. Chandrasekharan—H.R.B. Bulletin 44.

(2) Stabilisation of Lateritic Soils by A. Remillon, Director, Brazzaville Branch of the Laboratories for Construction and Public Works of the French Government, H.R.B., Bulletin 108.

(3) "Considerations Affecting the Design and Construction of Stabilised Soil Road Bases" by D. J. Maclean. Road Research Laboratory. D. S. I. R., U. K. The Journal of the Institution of Highway Engineers, Vol. III, No. 9, January 1956.

(4) Government of India, Ministry of Irrigation and Power Annual Report (Technical) 1952, Article on Soil Cement as a Construction Material.

(5) Journal of the Soil Mechanics and Foundations Division, A.S.C.E., Vol. 83. Article on "Stabilisation of Materials by Compaction" by W. J. Turnbull.

(6) Laboratory Manual in Soil Mechanics by Raymond F. Dawson.