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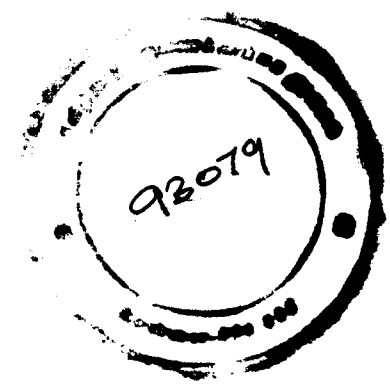
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“ RELATIVE COMPACTING EFFICIENCIES OF VIBRATION AND SMOOTH-WHEEL ROLLERS ”

By

Y. C. GOKHALE † & N. MOHAN RAO †

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SYNOPSIS

In order to evaluate the relative compacting efficiencies of vibration rollers and smooth-wheel rollers, the performance, in the field, of two types of vibration rollers and 6 and 10 ton smooth-wheel rollers was studied.

The compacting efficiencies of the rollers when used in (a) water-bound macadam on brick soling, (b) bituminous premix on water-bound macadam with and without seal coat, and (c) four different types of soils were determined. The compaction obtained was evaluated by means of plate bearing, field C.B.R. and field density tests except for water-bound macadam where the angle of inclination of the longest dimension of the stone pieces in the surfacing to the horizontal was taken to be an index of compaction. The method of measuring this angle is described.

1. OBJECT

The objects of the study were :

- (1) To compare the compacting efficiencies of vibration rollers with those of smooth-wheel rollers.
- (2) To tie the results of compaction obtained with vibration rollers with those of an earlier study* where the sheepsfoot, pneumatic and smooth-wheel rollers were used.

† Central Road Research Institute, New Delhi.

* Road Research Note No. 3 on “ The Compaction of Some Alluvial Soils of India,” published in the Journal of the Indian Roads Congress, Vol. XVII-2,

2. DETAILS OF CONSTRUCTION OF TEST SECTION

2.1. General Layout

Fig. 1 shows the general layout of the two experimental stretches constructed for the purpose. Each stretch was 10 ft wide and 140 ft long. One stretch was compacted with the flat roller. In the second stretch, a width of 6 ft was compacted with

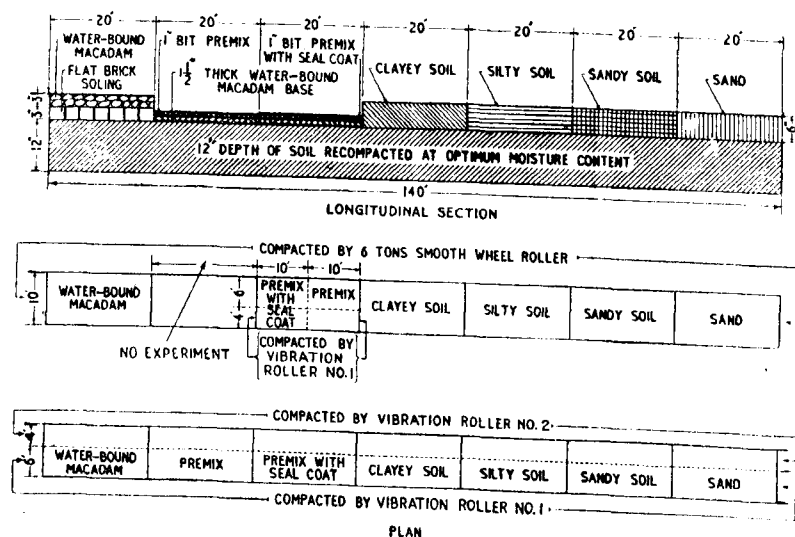


Fig. 1. Layout of the experimental stretches

vibration roller No. 1, para. 3.2, and the other four feet was compacted with vibration roller No. 2, para. 3.3. In experiments where soil is used, the passes of the rollers were confined to a width equal to the width of the roller. On each stretch the various specifications were laid in lengths of 20 ft each.

Each stretch consisted of roads laid to seven specifications, out of which one was water-bound macadam, one bituminous premix without seal coat, one bituminous premix with seal coat, and four were of different soils.

Construction Details of the Various Sections

2.2. Water-bound Macadam on Brick Soling

A single layer of flat bricks was laid over a compacted thickness of 12 in. layer of natural soil. Over the brick layer, the

water-bound macadam having a compacted thickness of 3 in. was laid. The mechanical analysis of the Delhi quartzite metal used for the water-bound macadam is given in Table 1.

TABLE 1

Mechanical Analysis of Delhi Quartzite used in Water-bound Macadam

	Per cent
Passing two inch sieve and retained on one inch sieve :	74
Passing one inch sieve and retained on half inch sieve :	12
Passing half inch sieve :	14

2.3. Bituminous Premix on Water-bound Macadam Without Seal Coat

A 1 1/2-in. thick water-bound macadam coat using a single layer of 1 1/2-in. size metal was laid over a 12-in. compacted thickness of natural soil. A tack coat of a cold application cut-back was applied at the rate of 20 lb per 100 sq. ft. over the water-bound macadam. The bituminous premix using 5 1/2 cu. ft. of 1/2 in. stone, 5 1/2 cu. ft. of medium sand, and 60.5 lb of cut-back 300/350 per 100 sq. ft. was then laid and rolled.

2.4. Bituminous Premix on Water-bound Macadam with Seal Coat

The thickness of water-bound macadam and the rate of application for the tack coat were the same as those described in para. 2.2. Bituminous premix using 10 cu. ft. of 1/2 in. size broken stone and 40 lb of 300/500 cut-back per 100 sq. ft. was laid uniformly over the tack coat and rolled. A seal coat with 2 1/2 cu. ft. of medium sand and 20 lb of 300/500 cut-back per 100 sq. ft. was spread uniformly on the premix and rolled.

2.5. Soil Sections

In order to compare the results obtained in this investigation with those obtained in the earlier work ¹, soils similar to those used in the previous work were selected. The analysis of the soils used in the present study together with those of the soils used in the previous study are given in Table 2. The details of the mechanical analysis of the four soils used in this study are given in Table 3.

TABLE 2
Characteristics of the Soils selected and the Soils used in the Study

Description of the Soil	Soil used in this study			Soil used in previous study		
	L. L.	P. I.	Sand content, per cent (above .05 mm.)	L. L.	P. I.	Sand content, per cent (above .05 mm.)
Clayey soil	31.40	13.10	38.40	35.12	15.75	32.00
Silty soil	26.80	10.10	52.50	30.00	10.80	52.00
Sandy soil	25.56	6.16	62.00	23.20	4.10	61.40
Jamuna sand	—Non-plastic—			—Non-plastic—		
			100.00			100.00

TABLE 3
Mechanical Analysis of Soils used in the Study

Name of Soil	Percentage of		
	Sand 2.00 mm. 0.05 mm.	Silt 0.05 mm. .005 mm.	Clay less than .005 mm.
Clayey soil	38	41	21
Silty soil	53	30	17
Sandy soil	62	24	14
Sand	100	—	—

Each soil was laid to a loose thickness of about 9 inches and rolled. In the case of clayey soil, fixing the passes of vibration roller No. 1 at 64, the optimum moisture content for the maximum density at that compactive effort was determined in the field. Similarly, for the corresponding compactive effort, the maximum density and optimum moisture for the silty and sandy soils were determined based on laboratory tests, para. 4.3. It was not possible to roll pure sand satisfactorily with vibration roller No. 1 and, as such, the tests were for clayey, silty and sandy soils only.

To have uniform foundation for all the specifications, the soil at site was excavated to a depth of 18 inches. The results of the physical tests on the natural soil at the site are given in Table 4.

TABLE 4
Physical Characteristics of the Natural Soil at the Site

	Percent
Liquid Limit	31.00
Plasticity Index	15.70
Sand content (2.00 mm.—0.05 mm.)	43.40
Optimum moisture content (Standard Proctor)	13.10

The excavated soil was pulverised to pass through 3/16 in. sieve and stacked. Each stack was 1 ft high and 100 cu. ft. in

volume. Based on the optimum moisture and an allowance of $1\frac{1}{2}$ per cent for evaporation losses, measured quantity of water was added to each stack on top and was allowed to soak through over night.

The next morning, the soil was thoroughly mixed and the moisture content was checked before spreading. The soil was then spread to a 9-in. loose layer, levelled and rolled with a 6-ton smooth-wheel roller, till the desired density was obtained.

The four soil test surfaces were laid directly over the natural soil of 12 in. compacted thickness.

3. ROLLERS USED IN THE STUDY

3.1. Six-ton Smooth-wheel Roller (Photo 1)

The 6-ton smooth-wheel roller is a three-wheeled diesel roller with rear wheels of 4 ft 3 in. diameter and 1 ft 6 in. width and a front wheel of 2 ft 10 in. diameter and 4 ft width.

3.2. Vibration Roller No. 1 (Photo 2)

Weight	1.6 tons
Diameter of cylinder	2 ft $5\frac{1}{2}$ in.
Width of cylinder	2 ft $11\frac{1}{2}$ in.

The roller is self-propelled and has two speeds, that is, 50 ft per minute and 100 ft per minute. The vibration frequency can be varied from 2,400 to 3,400 cycles per minute. The cost of the roller is about Rs 20,000.

3.3. Vibration Roller No. 2 (Photo 3)

Weight	4.25 cwt
Diameter of cylinder	1 ft 9 in.
Width of cylinder	2 ft

The roller is not self-propelled and has to be pulled by two men. A petrol engine of 1.3 horse power imparts vibrations at a fixed frequency of 4,500 cycles per minute.

For the water-bound macadam it was found that the compaction obtained by vibration roller No. 1 was higher than that obtained by 6-ton smooth-wheel roller. Hence with a view to compare the compaction obtained by vibration roller No. 1 with a heavier smooth-wheel roller, in addition to the 3 rollers described above, a 10-ton smooth-wheel roller was also used for consolidation.

4. FIELD EXPERIMENTS

4.1. Procedure Adopted for arriving at the Optimum Frequency of Roller No. 1 for the various Soil Specifications

As the vibration roller No. 1 had a frequency range of 2,400 to 3,400 cycles per minute, an attempt was made to determine the optimum frequency of the roller for the clayey and sandy soils used.

The soil under test was spread in a trench 32 ft long in stretches 8 ft long and 3 ft wide each to a depth of 9 in. loose at the optimum moisture content determined by the standard Proctor compaction method. Each of the four stretches was rolled with 64 passes of the roller with vibration frequencies of 2,400, 2,600, 3,000 and 3,200 cycles per minute, and the maximum dry bulk densities were determined. Fig. 2 shows the relationship between the frequency and the maximum dry bulk density, for the clayey and sandy soils.

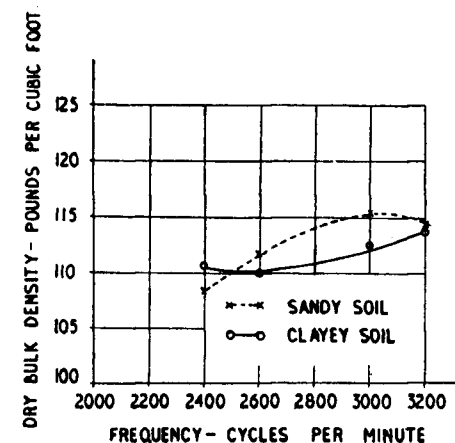


Fig. 2. Relation between the dry bulk density and frequency of vibration roller No. 1 on 9 in. loose layers of clayey and sandy soils

The maximum density for the sandy soil was obtained at a frequency of 3,000 cycles per minute whereas the density of clayey soil increased with frequency up to 3,200 cycles per minute and no optimum could be determined within the workable range of the machine. However, as the density obtained at 3,200 cycles was higher by only one lb per cu. ft. than

that with 3,000 cycles per minute, the optimum frequency was fixed as 3,000 cycles per minute for all the four soils.

4.2. Effect of Moisture Content on the Dry Bulk Density of Clayey Soil Compacted by the Vibration Rollers

Clayey soil, mixed with moisture contents varying from 7 to 15 per cent, was laid in stretches 5 ft long and 9 in. thick (loose) each, and was compacted by vibration rollers No. 1 and No. 2 at a frequency of 3,000 cycles per minute. The dry bulk densities of the compacted soil at different moisture contents were determined after 64 passes of the rollers. Fig. 3 shows the moisture-content dry-bulk-density relationship of the clayey soil when vibration roller No. 1, vibration roller No. 2

- Field compaction curve for vibration roller, No. 1.
- *---* Field compaction curve for vibration roller No. 2.
- Field compaction curve for smooth-wheel roller.
- △---△ Laboratory compaction curve simulating field compaction of vibration roller No. 1.
- ⊙---⊙ Laboratory compaction curve simulating field compaction of vibration roller No. 2.

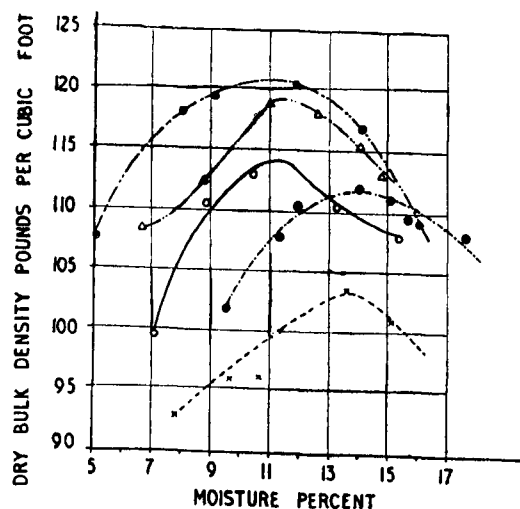


Fig. 3. Moisture content and dry bulk density of clayey soil with different compacting efforts

and 6-ton smooth-wheel roller were used. The results of laboratory compaction tests on clayey soils using the equivalent of field compactive efforts are also shown in the Figure. For the smooth-wheel roller the curve in Fig. 3 was plotted using the data obtained in the earlier investigations.

4.3. Determination of Laboratory Compactive Effort to Produce the Same Optimum Moisture Contents as Those Obtained in the Field with the Vibration Rollers

Fig. 3 indicates that in the clayey soil the optimum moisture contents for the vibration rollers No. 1 and 2 were 11.3 and 13.7 per cent respectively.

Laboratory compaction tests were carried out with the Proctor mould, for clayey soil, to find the compactive effort required to achieve the same optimum moisture contents as obtained in the field by varying the number of blows, the number of layers and the weight of the rammer. By trial and error, it was found that:

- (i) The optimum moisture content of clayey soil in the laboratory was 11.3 per cent when it was compacted in the standard proctor mould in five layers using 10 blows per layer with a 10-lb rammer and a drop of 18 inches. Hence this compactive effort was taken as the equivalent of field compaction obtained with the vibration roller No. 1
- (ii) The optimum moisture content of clayey soil in the laboratory was 14.0 per cent (as against the required 13.7 per cent) when the soil was compacted in the standard Proctor mould in three layers using 10 blows per layer with a 5½-lb rammer and a drop of 12 inches. Hence this compactive effort was taken as the equivalent of field compaction of the vibration roller No. 2. For determining the optimum moisture contents, the laboratory compactive efforts mentioned above were used for the silty and sandy soils also, thus eliminating detailed field trials.

The relation between the moisture content and the resulting density for the different soils with the different compactive efforts is shown in Fig. 4 and 5. The curve relating to smooth-wheel roller was drawn from the data obtained in the earlier study.

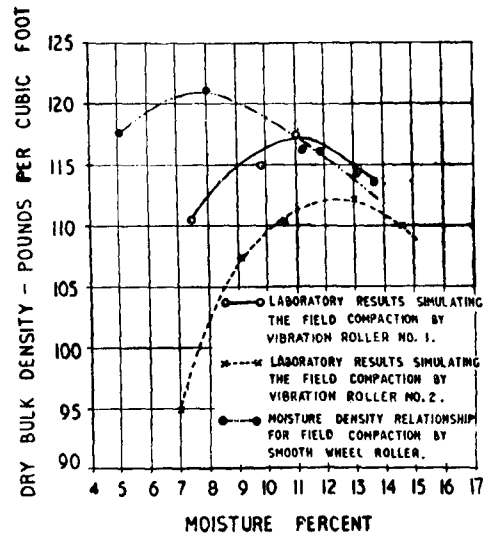


Fig. 4. Moisture content and dry bulk density for silty soil

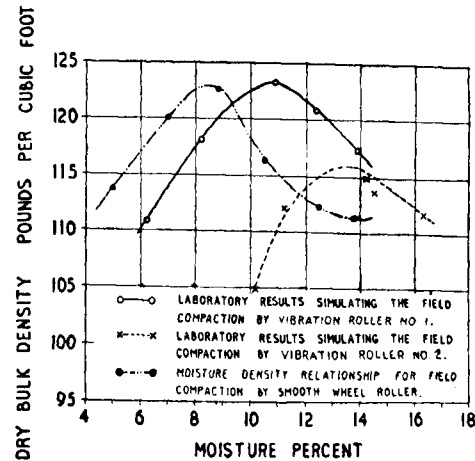


Fig. 5. Moisture content and dry bulk density of sandy soil

4.4. Variation in Density of Different Soils with increasing Number of Passes

The effect of number of passes of different rollers on the density of soils was determined by finding the field density before rolling and at the end of 2, 4, 8, 16, 32 and 64 passes while compacting with the various rollers. The moisture used in this experiment was maintained equivalent to the optimum moisture for the various rollers at 64 passes. Fig. 6, 7 and 8 show the relationship between dry density and number of passes of each roller for clayey, silty and sandy soils. For the sake of comparison, similar curves obtained in the earlier study for 7-ton smooth-wheel roller are also shown in Fig. 6 to 8.

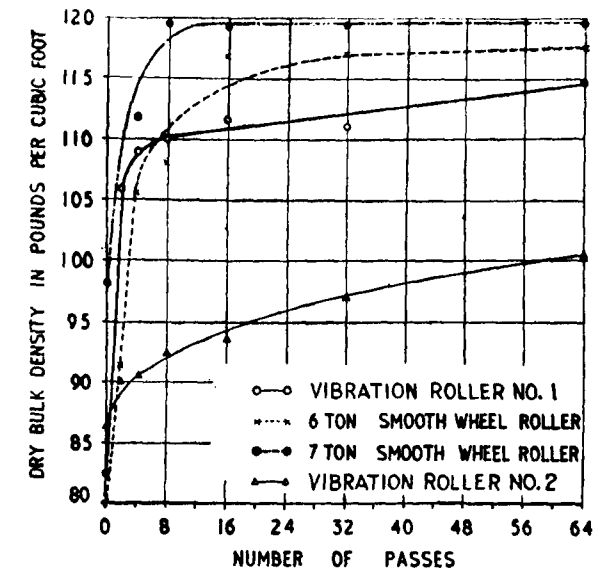


Fig. 6. Number of passes of the different rollers and dry bulk density of clayey soil compacted at the optimum moisture content for each roller in 9 in. loose layers

4.5. Procedure for Evaluation of Compacting Efficiency

Six ton smooth-wheel roller, vibration roller No. 1 and vibration roller No. 2 were used on strips of 10 ft, 6 ft and 4 ft in width respectively.

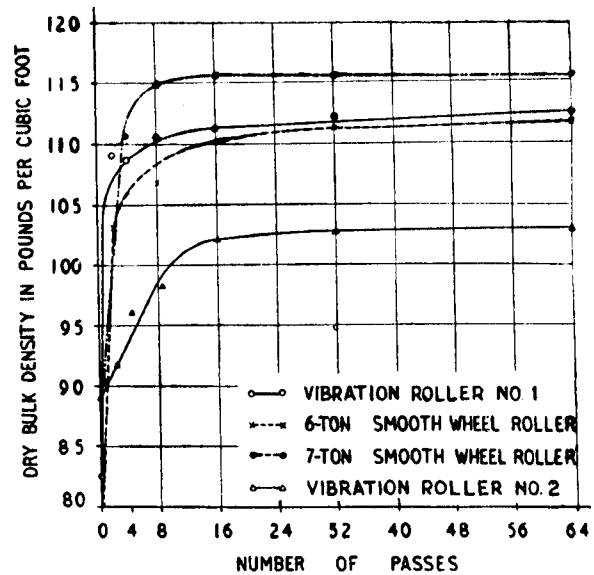


Fig. 7. Number of passes of the different rollers and dry bulk density of silty soil compacted at the optimum moisture content for each roller in 9 in. loose layers

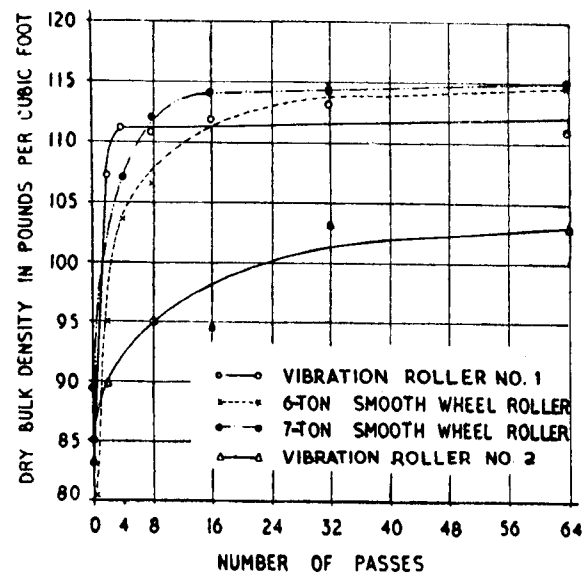


Fig. 8. Number of passes of the different rollers and dry bulk density of sandy soil compacted at the optimum moisture content for each roller in 9 in. loose layers.

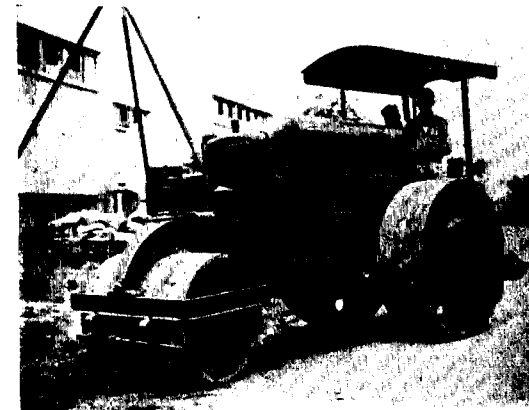


Photo 1. Six-ton smooth-wheel roller



Photo 2. Vibration roller No. 1

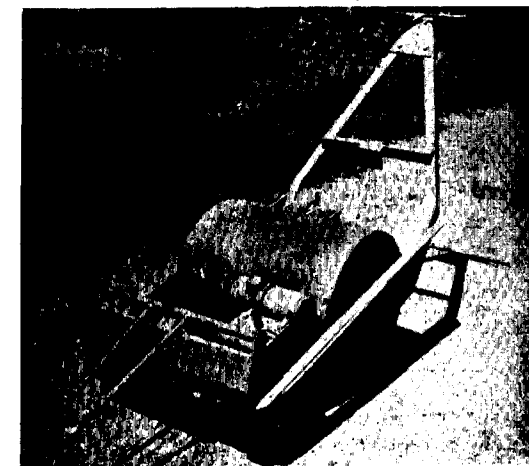


Photo 3. Vibration roller No. 2

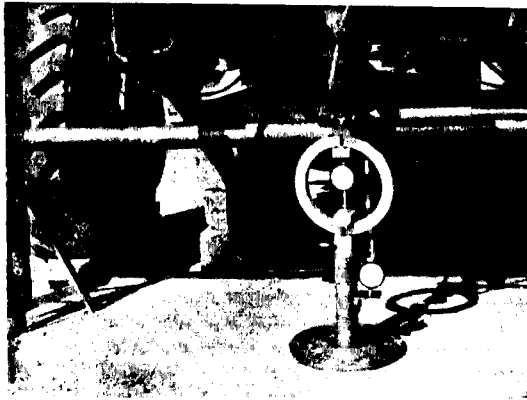


Photo 4. Set-up for field C. B. R. test



Photo 5. Set-up for plate bearing test

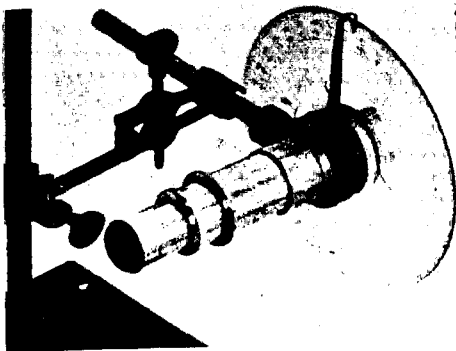


Photo 6. Instrument for measuring the angle of inclination of the longest dimension of stone pieces of water-bound macadam



Photo 7. The measuring of the angle of inclination of stone pieces in W. B. M.

The compaction produced by a roller depends on the pressure exerted by the roller and the number of passes it makes. As the purpose of the experiment was to compare, on the whole, the rolling efficiencies, the number of passes of the three rollers was kept proportional to the width of surfaces they had to roll, irrespective of the width of the roller.

The exact number of passes required for vibration roller No. 1 for satisfactory compaction, was first found out and the number of trips required to be made by other rollers for different experiments were calculated on the basis of the width of the experiments.

The number of passes of each type of roller for the various specifications are given in Table 5.

TABLE 5

Number of Passes made with each Type of Roller

Description of roller	Width of strip rolled in feet	No. of passes
Water-bound Macadam :		
6-ton Smooth-wheel Roller	10	87
Vibration Roller No. 1	6	52
Vibration Roller No. 2	4	35
Bituminous pre-mix with seal coat :		
6-ton Smooth-wheel Roller	6	57
Vibration Roller No. 1	4	38
Bituminous pre-mix without seal coat :		
6-ton Smooth-wheel Roller	6	30
Vibration Roller No. 1	4	20

5. FIELD TESTING

5.1. Field C. B. R. Test

The load was applied by means of a 4-ton mechanical jack and was measured by means of a 5-ton proving ring. The necessary reaction for the application of the load was obtained from a 6-ton loaded truck. The rest of the procedure followed for the test was as per common practice. Photo 4 shows the set-up for field C.B.R. test.

5.2. Plate Bearing

The load was applied by means of a 25-ton hydraulic jack, connected to a pump unit, and was measured on the calibrated dial of the pump unit. The necessary reaction for the application of the load was obtained from a 6-ton loaded truck. Due to the limited reaction conveniently available in the field, the plate bearing tests were conducted only on a 12-in. diameter plate. Two dial gauges were used to measure the settlement of the plate. The modulus of subgrade reaction presented in the data is calculated for a 12-in. diameter plate for 0.05 in. average settlement. The rest of the procedure followed was as per the standard practice. Photo. 5 shows the set-up for plate bearing test.

5.3. Field Density Test

Soil Specification

In the case of soil specifications, the field density obtained in the various stretches was determined by the standard sand replacement method.

Bituminous Premix

For bituminous premixes, samples were cut and removed from the rolled surface and the densities got by the use of different rollers were determined. After determining the weight of the sample the amount of voids in the volume of the sample was found by water displacement method.

5.4. Water-bound Macadam Specification

The relative degree of compaction of the water-bound macadam was determined by measuring the average angle of inclination of the longest dimension of the pieces of stone as seen in a cut section. The instrument used for the measurement is described in para. 5.5.

5.5. Instrument for measuring the angle of inclination of aggregate particles

The instrument, Photo 6, was fabricated in the Central Road Research Institute. It, Fig. 9, consists of the tube of an ordinary travelling microscope fixed to a stand by means of a clamp. The object piece of the microscope is detached. The aggregate particle 'A' is viewed from the object piece end, G, through the eye-piece lens 'B' of the microscope.

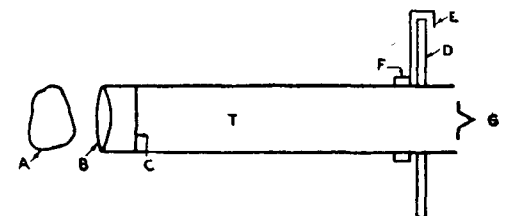


Fig. 9. The apparatus for measuring the angle of Inclination of pieces of aggregate

The tube can be rotated in the clamp *F*. A steel pointer, *E*, is fixed to the clamp *F*. The pointer indicates the angle in degrees on a circular protractor, *D*, which is fixed to the microscope tube. To avoid confusion, only one cross-wire, *C*, was retained.

5.6. Procedure for Measuring the Average Angle of Inclination

A 2-ft wide longitudinal section vertical cut was made such that about 25 stone pieces were clearly visible. The surface was cleaned of all loose pieces and the external face of the cut section was well cleaned with a piece of wet cloth. The instrument was placed in front of the cut section and the observer took readings lying flat on the ground. Photo 7 shows the measurements in progress.

To find the angle of inclination of the longest axis of the individual stone, the instrument was so adjusted that the entire stone under measurement was clearly seen through the lens, *B*. By visual inspection it was easy to identify the longest axis of the stone straightaway.

The tube which was always kept horizontal was then rotated so that the cross wire, *C*, coincided with the longest axis of

the stone as seen through the instrument and the angle measured was noted as 'x'. A plumb bob was held in front of the lens of the tube which was rotated in such a way that the cross wire coincided with the string of the plumb bob, giving an angle 'y' on the protractor. The angle between the longest axis and the horizontal was then worked out from the values of 'x' and 'y' by subtracting the value of ('y' - 'x') from 90 degrees. From the results obtained of the individual stone pieces involved in each test section, the root mean square value was calculated. The root mean square value was used for comparison of the compacting efficiency of the different rollers used.

5.7. Test Results

The results obtained for the various tests described are given in Tables 6 and 7.

6. DISCUSSION OF TEST RESULTS

6.1. General

For purpose of evaluating the compacting efficiency of the vibration rollers, it is believed that the maximum compaction can be expected for a particular material only when the frequency of the compacting machinery is equal to the natural frequency of the material under compaction. Since the vibration roller No. 2 was of fixed frequency, it was not possible to arrive at the optimum frequency. The highest frequency of 3,200 cycles per minute of vibration roller No. 1 was below the natural frequency of the clayey soil that was tested.

The standard laboratory compaction test is based on the average compaction that is expected to be obtained with the average construction equipment. For determining the optimum moisture for compaction with different rollers having different compactive efforts, use of the standard laboratory compaction test is not justified. But knowing the optimum moisture and maximum density of any soil in the field for any given compacting machinery, it is possible to determine the equivalent laboratory compactive effort to produce the same optimum moisture by trial and error.

That vibration roller No. 2 is suitable only for light compaction was confirmed by the experiments. So, while studying the performance of plant for water-bound macadam and bituminous surfaces, the comparison was limited to the vibration roller No. 1 and the 6-ton smooth-wheel roller.

TABLE 6

Angle of inclination of the Aggregate Pieces with the Horizontal in the Water-bound Macadam Surface Compacted by different Types of Roller

Sr. No.	Angle with the horizontal in degrees			
	Vibration Roller No. 1	Vibration roller No. 2	6-ton smooth-wheel roller	10-ton smooth-wheel roller
1.	45	56	68	34
2.	20	2	50	50
3.	0	27	52	41
4.	80	52	70	5
5.	54	44	21	—
6.	51	18	80	37
7.	22	71	38	41
8.	20	38	88	11
9.	10	90	10	39
10.	28	38	10	43
11.	25	11	0	35
12.	8	87	24	55
13.	33	22	5	2
14.	43	71	16	42
15.	25	41	66	52
16.	13	69	53	52
17.	21	70	52	8
18.	41	17	25	10
19.	1	77	17	28
20.	71	12	26	54
21.	6	17	37	50
22.	7	46	25	10
23.	43	—	77	38
24.	8	—	25	33
25.	35	—	45	—
26.	25	—	7	—
27.	—	—	10	—.

TABLE 7

Maximum Densities and Optimum Moisture Contents obtained in the Field and in the Laboratory for different Soils and Test Values for Comparison of Compacting Efficiencies

Description of Roller used	Clayey Soil				Silty Soil				Sandy Soil			
	Maximum dry density gm per cc.	C.B.R.	Modulus of sub-grade reaction K lb per cu. in.	Optimum moisture content per cent	Maximum dry density gm per cc	C.B.R.	Modulus of sub-grade reaction K lb per cu. in.	Optimum moisture content per cent	Maximum dry density gm per cc	C.B.R.	Modulus of sub-grade reaction K lb per cu. in.	Optimum moisture content per cent
1. Vibration Roller No. 1 (64 passes)	1.83	26	953	11.5	1.80	30	823	10.5	1.80	23	1324	11.0
2. 10-ton Smooth-wheel Roller												
3. 6-ton Smooth-wheel Roller (64 passes)	1.88	52	1424	11.0	1.79	32	471	9.0	1.83	21	1444	8.0
4. Vibration Roller No. 2 (64 passes)	1.61			14.0	1.64			13.5	1.64			12.8
5. 7-ton Smooth-wheel Roller* (64 passes)	1.91			11.0	1.85			9.0	1.85			8.0
6. Pneumatic-tyred Roller* (64 passes)	1.78			12.0	1.80			8.3	1.72			11.0
7. Sheepfoot Roller (64 passes)	1.75			11.0	1.64			12.0	1.65			11.0
8. B. S. Laboratory Compaction Test	1.86			13.3	1.88			11.8	1.86			10.4
9. Laboratory Compaction Effort equivalent to Vibration Roller No. 1	1.91			11.5	1.98			10.5	1.88			11.0
10. Laboratory Compaction Effort equivalent to Vibration Roller No. 2	1.80			14.0	1.85			13.5	1.80			12.8

* From the Journal of the Indian Roads Congress, Vol. XVII-2.

TABLE 7 (Contd.)

Description of Roller used	Water-bound Macadam				Bituminous Premix			
	Angle of inclination of the longest axis to the horizontal. Root Mean Square		Modulus of sub-grade reaction K lb per cu. in.		With Seal Coat		Without Seal Coat	
	Degrees	Minutes	C.B.R.	Modulus of sub-grade reaction K lb per cu. in.	Field density in grams per cc	C.B.R.	Field density in grams per cc	Modulus of sub-grade reaction K lb per cu. in.
1. Vibration Roller No. 1 (64 passes)	30	50	81	1244	2.17	33	2.37	51
2. 10-ton Smooth-wheel Roller	36	42	78	1240				1304
3. 6-ton Smooth-wheel Roller (64 passes)	44	43	57	783				
4. Vibration Roller No. 2 (64 passes)	51	26	...	400	2.28	41	2.48	27
						663		1866

6.2. Methods Adopted for Evaluating the Compacting Efficiency of the Various Rollers

Field Density Test

As long as the moisture content is the same for the particular type of soil, it is reasonable to say that the density is a direct measure of the void content which in turn is directly dependent upon the degree of compaction. Hence the dry density is taken as a direct measure of the degree of compaction obtained with the various rollers.

6.3. Soil Specifications

Fig. 6 to 8 indicate that the density obtained with the 7-ton smooth-wheel roller is slightly higher than the corresponding density obtained with the 6-ton smooth-wheel roller, the difference being about 3 lb per cu. ft. As this difference is not significant, when compared to the differences obtained in the earlier study within the same specification, it is considered justifiable to tie up the results obtained in this study for the vibration rollers with those obtained by other rollers in the earlier study. Fig. 7 and 8 indicate that there is no significant difference in the compaction obtained with vibration roller No. 1 and the two smooth-wheel rollers for silty and sandy soils. But in clayey soils the density obtained by vibration roller No. 1 is slightly less than the densities obtained with the smooth-wheel rollers. The compaction obtained with the vibration roller No. 2 in all the three soils is considerably less than the compaction obtained with the vibration roller No. 1 and the 6 and 7-ton smooth-wheel rollers.

6.4. Bituminous Specifications

Table 7 indicates that the density obtained with vibration roller No. 1 is slightly higher than that obtained with 6-ton smooth-wheel roller in the case of bituminous premix without seal coat, whereas in the case of bituminous premix with seal coat, the density obtained with 6-ton smooth-wheel roller is slightly higher than that obtained with vibration roller No. 1. It may be said that the difference in the performance of the two types of rollers is not significant in the case of bituminous specifications covered by this study.

6.5. Field C.B.R. and Plate Bearing Tests

Variation in moisture contents used in different specifications for obtaining the maximum density is bound to influence the strength values obtained by the C.B.R. and plate bearing

tests. As the same moisture content was not used for the various specifications rolled with different rollers, it is not possible to strictly interpret the results of these tests with the corresponding field densities for the purpose of evaluating the degree of compaction. But as it was difficult in the field to bring the various experimental lengths compacted at different optimum moisture contents to the same moisture content, the optimum moisture content was kept as a common base for purpose of evaluation of the strength tests.

The results of field C. B. R. and plate bearing tests for the various specifications given in Table 7 are not consistent in most of the cases. The period of field testing was unfortunately prolonged due to intermittent rains. And in some cases it was also not possible to conduct both C. B. R. and plate bearing tests simultaneously. It is presumed that these factors might have contributed to the apparent inconsistencies in the test results.

In the bituminous premix without seal coat, the results of field C. B. R. obtained agree with the density observations. That the performance of vibratory roller No. 1 is slightly better than 6-ton smooth-wheel roller in bituminous premix without seal coat is confirmed by this.

In water-bound macadam, the results of plate bearing tests indicate that the compacting efficiency of vibration roller No. 1 is higher than that of 6-ton smooth-wheel roller and is equal to that of 10-ton smooth-wheel roller. This was further confirmed by the angle measuring test.

6.6. Angle Measuring Test

The method adopted here for determining the degree of compaction of water-bound macadam surface is based on the principle that the smaller the angle of inclination to the horizontal, the greater the interlocking and degree of compaction. The root mean square value of the angle of inclination of the longest axis to the horizontal is taken as an index of the degree of compaction. Table 6 gives the angle of inclination of individual stones measured from cut sections rolled by various rollers. Based on the 'K' values and the values of the angles measured it could be seen that the performances of vibration roller No. 1 and 10-ton smooth-wheel roller are alike and that the performance of vibration roller No. 1 is better than that of 6-ton smooth-wheel roller in the case of water-bound macadam.

7. CONCLUSIONS

Under the conditions and limitations of the study, the following seems to be the trend of indications:

- (a) For compacting water-bound macadam, vibration roller No. 1 compares favourably with the 10-ton smooth-wheel roller, and is better than the 6-ton smooth-wheel roller.
- (b) For bituminous premix surfaces, there is no significant difference between the vibration roller No. 1 and the 6-ton smooth-wheel roller.
- (c) For compacting silty and sandy soils, there is no significant difference in the compactive performance of vibration roller No. 1, the 6-ton and 7-ton smooth-wheel rollers.

ACKNOWLEDGMENT

The Authors wish to express their gratitude to the Director, Central Road Research Institute for giving encouragement. The Authors are also greatly indebted to Shri C. G. Swaminathan for his valuable suggestion in connection with evaluation of compaction in water-bound macadam. Authors are specially indebted to Shri R. Krishnamachari and Shri H. S. Bhatia for their help. Acknowledgments are due to the members of Roads, Soils and Bitumen divisions for their co-operation at various stages of the work.

EFFECT OF KANKAR ON COMPACTION AND STABILITY OF SOIL

By

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AND

J. P. BHATNAGAR*

[Following precedent the research note received for publication was circulated to the Members of the Research Organization Division. The note, comments received, and the replies of the Authors were considered at the meeting of the Research Organization Division at Delhi on 12-2-57. The note with the comments, replies of the Authors and the minutes of the meeting are published for the information of Members.]

1. INTRODUCTION

A soil-kankar mixture was proposed for use as a wearing course in the construction of a low cost road in Lucknow district, Uttar Pradesh. The laboratory investigation described in this Note was undertaken to study how kankar would affect the compaction and stability of soil.

2. SOIL USED

The soil used in the investigation was collected from the Bani Harauni road and clay to be used as admixture was taken from a nearby pond. The admixture was adjusted to conform to the specification for wearing course suggested by Mehra¹. The characteristics of the soil are given in Table 1.

TABLE 1
Characteristics of the Soil

Liquid limit	28.9
Plastic limit	17.5
Plasticity index	11.4
Clay (particles smaller than 0.002 mm) per cent	25.0
Silt (particles between 0.002 mm to 0.02 mm) per cent	31.0
Sand (particles between 0.02 mm to 2 mm) per cent	44.0
Material retained on 200 mesh sieve, per cent	35.2
Textural classification	clay loam
Specific gravity	2.66
Maximum dry density in lb per cu. ft.	105.00

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The soil was found to belong to group A-6 of the classification adopted in the U.S.A. Public Roads Administration and contained more than 35 per cent silty clay materials passing No. 200 sieve.

3. AGGREGATE USED

The kankar was the one generally used in the road work. It was washed, dried, crushed, sieved and graded. The grading adopted is given in Table 2.

TABLE 2
Grading of the Aggregate

	per cent
Passing 1 in. sieve	100
„ $\frac{3}{4}$ in. „	85
„ $\frac{3}{8}$ in. „	65
„ $\frac{3}{16}$ in. „	50
„ 10 mesh	35

Specific gravity and water absorption were determined by the standard A.S.T.M. methods² and the results obtained were as follows :

Specific gravity = 2.77
Water absorption = 3.2 per cent

4. COMPACTION CHARACTERISTICS

4.1. The compaction test was carried out according to the A.S.T.M method³, with the only difference that the material was not passed through sieve No. 4 and the soil-kankar mixtures used contained graded kankar mentioned in Table 2. The standard mould of 4 in. internal diameter and a height of approximately 4.6 in. with a capacity of $1/30$ cu. ft. was used with a 5.5 lb hammer which was dropped from a height of 12 inches. The moisture determinations were carried out by drying the whole of wet sample of soil-kankar mixture each time after compaction instead of small sample of the mixture as it was not possible to

obtain a representative sample. The optimum moisture and the maximum dry density of the mixtures are given in Table 3. The moisture-density relationships are shown in Fig. 1.

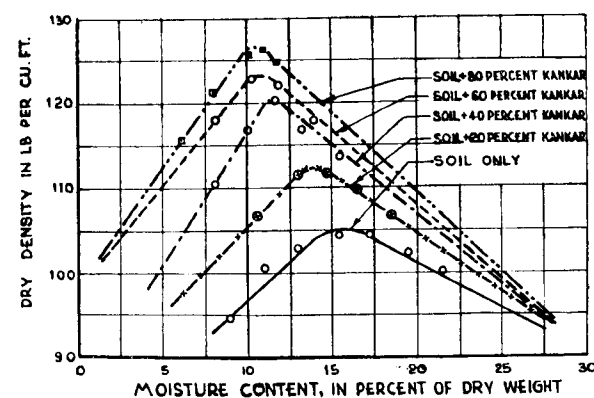


Fig. 1. Maximum dry density and optimum moisture relationship of various soil-kankar mixtures

TABLE 3
Maximum Dry Density and Optimum Moisture of Soil
with different Percentages of Kankar

Composition of mixture	Optimum Moisture content per cent	Maximum dry density lb per cu. ft.
Soil only	16.0	105.00
Soil + 20 per cent kankar	14.3	111.90
Soil + 40 per cent kankar	12.0	120.80
Soil + 60 per cent kankar	11.0	122.90
Soil + 80 per cent kankar	10.6	126.50

4.2. From Fig. 1, it is observed that there is increase in the dry density with increase in the addition of kankar. This might

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be due to the greater unit weight of kankar than that of the soil, with no effect on the maximum dry density or other properties of the soil portion of the mixture. Further, it is noticed that there is a decrease in moisture content at maximum density with the addition of increased amounts of kankar to the soil. This decrease in moisture content with increased amount of kankar is perhaps due to less quantity of moisture (3.2 per cent) required to lubricate the bigger kankar particles.

4.3 As the addition of kankar has changed the values of maximum density and optimum moisture content of the soil with which it is mixed, it is necessary to carry out the compaction test in the laboratory on the material containing all fractions of coarse aggregate as used in the field.

5. STABILITY OF SOIL-KANKAR MIXTURES

5.1. Norman W. McLeod⁴ outlined the use of triaxial compression test for determining the stability of granular and cohesive materials. He has suggested that stability of the materials depended on their capacity for carrying a greater applied vertical load, V , for a given lateral support, L . Consequently, for a specified value of lateral support, L , the most stable material is that for which the value of $(V - L)$ is the greatest for the equilibrium conditions established by the Mohr envelopes for the materials under comparison.

In the present investigation, the triaxial test on remoulded soil-kankar mixtures containing 0, 20, 30, 35, 40, 45 and 60 per cent of graded kankar was employed to study the stability of soil-kankar mixtures with 0, 5, 10 lb per sq. in. lateral pressures.

5.2. Specimens were made at optimum moisture and maximum density in the split cylindrical constant volume mould which had a known internal volume (8 in. long and 2.8 in. diameter when fitted with plungers). A predetermined weight of soil-kankar mixture with moisture content corresponding to maximum density was taken and thoroughly mixed. Wet soil-kankar mixture was added in 8 approximate equal instalments in the mould tamping every time with a light tamper weighing 1 lb to ensure uniform compaction. Finally, the top and bottom plungers were pressed in a compression machine to have specimens of constant volume and of required density at optimum moisture. The mixing of soil and making of the blocks were done in a constant temperature room at 25° C. in order to keep the evaporation

losses uniform. The specimens were enclosed in rubber sleeves and were subjected to triaxial test⁵ immediately.

5.3. The results are plotted in Fig. 2. It is concluded that maximum stability value, $V - L$, is attained when 35 per cent of kankar is mixed with soil. When the percentage of kankar is

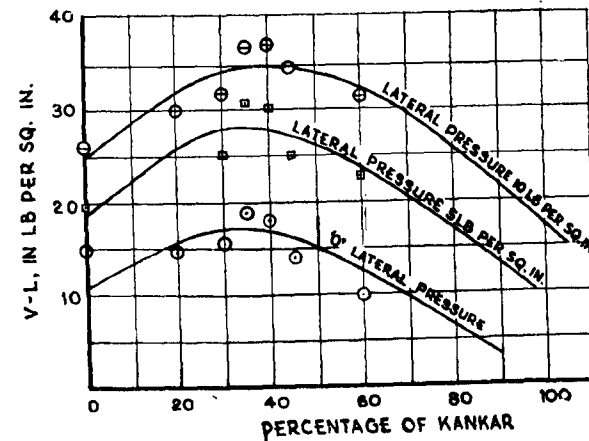


Fig. 2. Stability of soil kankar mixtures, $V-L$ at different lateral pressures

increased to 40 per cent, the stability is practically the same as that obtained with 35 per cent. On further addition of kankar the stability value, $(V - L)$, goes on decreasing.

5.4. Mohr diagrams, Fig. 3, 4, 5 and 6, were drawn for soil and for the different soil-kankar mixtures and the values of

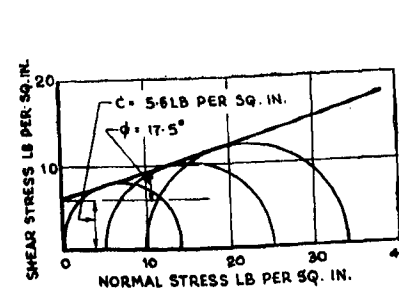


Fig. 3. Mohr diagram for soil

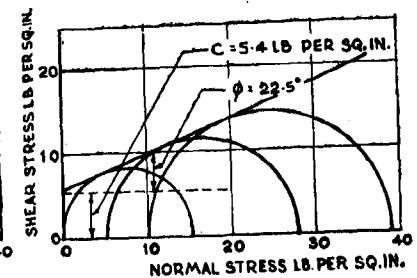


Fig. 4. Mohr diagram for soil containing 20 per cent of kankar

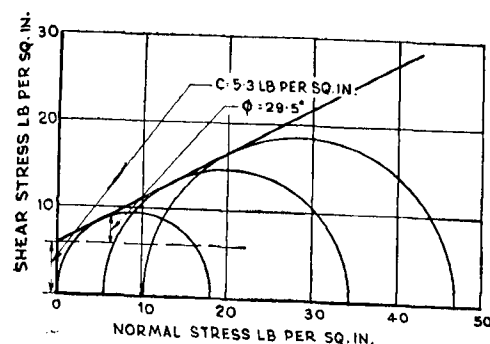


Fig. 5. Mohr diagram for soil containing 40 per cent of kankar

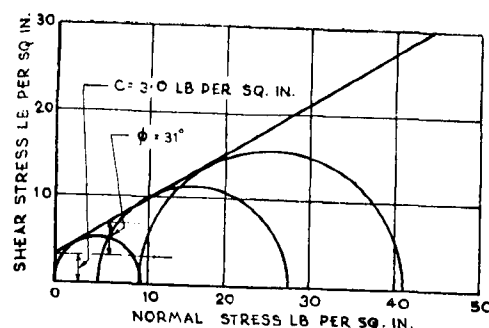


Fig. 6. Mohr diagram for soil containing 60 per cent of kankar

cohesion and angle of internal friction were determined. These are given in Table 4.

TABLE 4

Values of Cohesion and Angle of Internal Friction
for Different Soil-Kankar Mixtures

Composition of the admixture	Cohesion, C , in lb per sq. in.	Angle of internal friction, ϕ , in degrees
Soil only	5.6	17.5
Soil+20 per cent kankar	5.4	22.5
Soil+40 per cent kankar	5.3	29.5
Soil+60 per cent kankar	3.0	31.0

From Table 4 it is observed that the angle of internal friction increases with increasing percentages of kankar. There is, however, not much decrease in the value of cohesion for mixtures containing up to 40 per cent of kankar. Addition of kankar beyond 40 per cent reduces the cohesion considerably.

6. CONCLUSIONS

6.1. The addition of kankar to soil increases the maximum dry density of the soil-kankar mixture and decreases its optimum moisture content.

6.2. It is necessary to carry out the compaction test on the material containing all the fractions used in the field.

6.3. For maximum stability, the percentage of kankar aggregate should not be less than 35 per cent and not more than 40 per cent.

6.4. It has been observed that under conditions of maximum density and optimum moisture, the angle of internal friction of soil goes on increasing with the addition of increased percentages of kankar, whereas there is no marked decrease in the value of cohesion up to 40 per cent addition of kankar. Further addition of kankar decreases the value of cohesion considerably.

7. ACKNOWLEDGMENTS

The investigation was carried out in the Uttar Pradesh, P.W.D. Research Institute, Lucknow, and is published with the kind permission of the Chief Engineer.

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COMMENTS

Shri K. F. Antia remarked that the progressive increase in the density of soil-aggregate mixture with increase in the aggregate content was contrary to the results of certain other investigations abroad. Dr. Maddison has reported⁶ that the dry density of soil aggregate mixture as a whole increased up to a maximum stone content of 40 per cent; at stone contents higher than about 70 per cent, the individual stones were in contact with one another and so no further compaction of the soil mortar was possible. Results obtained in a later work in the Road Research Laboratory at Harmondsworth (U.K.) were also similar to those reported by Maddison.

It would seem reasonable to expect that, on addition of coarse aggregate to a cohesive soil beyond an optimum, there would be resistance to compaction of the soil mortar due to the interlocking of the aggregates resulting in reduced density of the mixtures. The progressive increase in density observed in the present investigation was best explained as due to the crushing of aggregate particles. In that event, the grading of kankar would appear to be of no importance.

Lateral pressures up to a maximum of 10 lb per sq. in. alone were tried in the test. Using the values of the angle of internal friction and cohesion obtained for the various soil and kankar mixtures, it would be found that for a lateral pressure of 50 lb per sq. in., the value of $V-L$ is greater for soil with 60 per cent kankar than for soil with 40 per cent kankar. This indicated that the former was the stabler mixture. The amount of lateral support which can be developed by a base course was largely unknown. The method employed by the Authors for choosing the most stable mixture did not seem to be applicable unless the maximum lateral pressure that would be developed in the base course was known.

The strength of a soil and its shearing resistance increased with increasing dry density. Hence the mixture which had the highest dry density would be the most stable one as well.

The specimen employed in the test was 2.8 in. in diameter and 8 in. high. It was desirable that the diameter of the specimen be five times the largest particle size since the maximum particle size was 1 in. Specimens larger than 5 in. diameter and 14 in. high should have been used in the tests.

Shri J. S. Narasimham observed that the problem investigated by the Authors was of much practical importance. The successful completion of the investigation would contribute the

necessary criteria required for the design of large mileage of rural highways which, for several years to come, would have to be soil stabilized kankar or gravel roads.

The kankar aggregates used in the investigation as the stabilizing agent should have been tested for Los Angeles Abrasion, and the test data used in predicting the per cent of kankar that would be degraded under the anticipated traffic intensity. It had been well established that "wet rolling" of disintegrated kankar developed cementitious properties. So, the fraction which got degraded under traffic contributed to the cohesion of the soil. Therefore, the fraction of the kankar metal likely to be disintegrated under the traffic intensity should be ascertained and taken into consideration in the fixation of the optimum per cent of kankar as a stabilizing agent.

The gradation analysis of the stabilized soil mixture with different percentages of kankar aggregates compacted to the respective optimum densities would give valuable information for fixing the optimum percentage of the stabilizing agent. If, instead of kankar, coal ash were to be used as the stabilizing agent the determination of optimum density would be a difficult affair as the influence of degradation of the cinders under the compaction would affect the density. The same concept held good to a certain extent for the kankar used in this investigation.

The Authors, while admitting that an increase in the admixture of kankar had resulted in an increase in the density of the soil mixture, had concluded that further increase in admixture produced "no influence" on the soil portion of the mixture. This conclusion appeared to be based on no factual evidence.

An increase in the admixture over 80 per cent will completely change the nature of the investigation, as the resulting mixture instead of being soil stabilized with kankar would result in a clay bound kankar" macadam surface. If the percentage of the admixture was increased to 100, the density obtained would be greater than that obtained with 80 per cent. Also, the percentage of clay fraction would increase due to the degradation of the kankar aggregates under the dynamic compaction used in the Proctor test. The "Theoretical Maximum Density" at zero air voids at different percentages of admixtures could easily be determined and this would prove that density would increase with increase in admixture of kankar. Shri Narasimham was of opinion that admixtures up to 100 per cent of the soil should be investigated. In his opinion, mixtures more than 100 per cent of kankar would result in a change in the nature of the problem.

It is preferable to conduct in this investigation "open type triaxial" tests to simulate the field conditions. The equation given by McLeod for stability of materials with both granular and cohesive properties was:

$$V - L = \frac{2L \sin \phi}{1 - \sin \phi} + 2C \frac{1 + \sin \phi}{1 - \sin \phi} \dots\dots\dots (1)$$

Also, he had correctly pointed that the equation would not be satisfactory for the design of surfacings as it would permit the use of a material with zero cohesion. He had recognised that for a given value of lateral pressure, L , the stability of the material, $(V - L)$, increased as C or ϕ or both increased. The amount of lateral support, L , which can be developed by a base course is at present unknown and the methods followed by different authorities to determine this value were still empirical. The values of C and ϕ for a given material could be determined in the laboratory by triaxial tests. The normal wheel load expected on the surface was known. Hence, from the stability equation given by McLeod, the value of L can be determined for given value of C , ϕ and V . Till such time that the value of L developed can be correctly determined by experimental methods, some values based on field observations had to be assumed and the values of V from equation (1) should be calculated for the given values of C and ϕ of the soil. Having obtained the values of V , the design of the base course would then be simple and the percentage of the stabilizing agent could be determined.

There are certain limitations in taking the value of $V - L$ as a stability yardstick for the mixtures. McLeod in his Paper had mentioned that the stability value, $V - L$, should be fixed taking the minimum desirable values of C and ϕ for the material into consideration. A reference to Table 4 indicated that at admixture of 35 to 40 per cent the cohesion was found to be about 5.3 lb per sq. in. and angle of internal friction was 29.5°. It was open to question whether these strength values for the mixture would give a suitable road surface. These values would considerably be reduced when the sample was tested under saturation conditions. While the theory advanced by McLeod deserves commendation, the interpretation of the triaxial results and the use of values of $V - L$ as a stability yardstick deserves careful consideration.

It was generally believed that increase in density would result in increase in compressive strength. The desirable density and the minimum voids in the compressed mixtures should be fixed and the percentage admixture of kankar selected accordingly.

Valuable information could have been collected if the Authors had conducted the C.B.R. tests on these admixtures and compared the data obtained with the relationship shown in Fig. 2 of the Note. Although the C.B.R. method was also semi-empirical, road surfaces designed on the basis of C.B.R. had given satisfactory service performance.

AUTHORS' REPLY

Increase of dry density of soil-kankar mixture up to the addition of 80 per cent of graded kankar aggregate were not contrary to the results reported abroad, as mentioned by **Shri K. F. Antia**.

Maddison⁶ found that the dry density of the soil-stone mixture as a whole increased with increasing amounts of stone up to a stone content of 40 per cent. Graded aggregate $\frac{3}{4}$ in. to No. 7 B.S. (2 mm diameter opening) was used by the Road Research Laboratory, Harmondsworth (U.K.) and it has been reported⁶ that density of soil-aggregate mixture increased with increasing percentage of aggregate up to 50 per cent.

Zeigler⁷ studied the effect of addition of gravel graded from $\frac{3}{4}$ in. to 4 mesh on soil and has reported that the dry density of soil-gravel mixture increased with the addition of gravel up to 50 per cent. But the effect of further addition of gravel had not been reported.

In the present investigation 35 per cent of the aggregate used passed through No. 10 A.S.T.M. sieve. Therefore, the material passing through No. 10 sieve may be treated as soil portion as is usually done in the mechanical analysis of soils and the remaining portion retained may be treated as aggregate. The addition of 80 per cent of graded aggregate in the present investigation corresponds to 52 per cent of aggregate $\frac{3}{4}$ in. to No. 10 (A.S.T.M.). Thus the increase of dry density observed in the present investigations up to 80 per cent addition of graded aggregate was in conformity with the investigations carried out abroad.

A base course failure had been reported by Hogentogler (Proceedings of the 4th Texas Conference) where there was deficiency in one of the sand fractions. So, in the graded kankar mixture 35 per cent of material passing No. 10 A.S.T.M. was included because it was necessary for stability.

Kankar was a weak aggregate as compared to stone or gravel and there was possibility of its being crushed under compaction to a grading closely approaching that required for maximum density. But for a mechanically stabilized mixture, it was necessary to have a well graded mixture.

In the present investigations the values of $V-L$ were obtained by the formula $V-L = \frac{P}{A} \frac{(1-\delta h)}{H}$, the values of P and δh having been got by triaxial compression tests at three different lateral pressures, namely, 0, 5 and 10. It was presumed that the Mohr's envelope for a particular soil-aggregate mixture remained the same for various values of lateral pressures. So, values of C and ϕ obtained by the tests were not affected even if the lateral pressure was increased to 50 lb per sq. in. Whereas McLeod had evaluated the value of $V-L$ based on the values of C and ϕ , experimental values of $V-L$ obtained for different mixtures at three different lateral pressures, namely, 0, 5 and 10, by triaxial apparatus were used in this investigation.

The data obtained was statistically analysed and used to get the best possible curves fitted. The equations for the curves shown in Fig. 2 of the Note so obtained were as under :

(i) For lateral pressure of 0 lb per sq. in.

$$Y = 10.67 + 0.356 x - 0.0054 x^2$$

(ii) For lateral pressure of 5 lb per sq. in.

$$Y = 18.87 + 0.486 x - 0.0096 x^2$$

(iii) For lateral pressure of 10 lb per sq. in.

$$Y = 24.80 + 0.471 x - 0.00585 x^2$$

The strength of soil usually increases with dry density and the most stable mixture may not have the highest dry density. The curves in Fig. 9.12 and 9.14 of Soil Mechanics for Road Engineers, showing relations between unconfined compressive strength, moisture content and dry density and between C. B. R. values, moisture content and dry density, indicate that in order to obtain maximum unconfined compressive strength (half of which is shear strength) or maximum C. B. R. value (which is an indirect measure of shear strength of soil), it is desirable to compact a soil at a moisture content less than the optimum (i. e. at a lesser density than the maximum dry density (para. 9.37, p. 169 of Soil Mechanics for Road Engineers).

The mould, 2.8 in. diameter and 8 in. height, was usually used for determining the shear characteristics of coarse grained soils. Sizes of specimen other than the one mentioned above could not be used in the shear apparatus.

In reply to Shri J. S. Narasimham, the Authors mentioned that the kankar used in the investigation was found to have an

abrasion value of 30 per cent by Los Angeles abrasion test. When a water-bound macadam was made of kankar, the angular shape of the kankar aggregates was lost by attrition among the particles due to vibrations set up by traffic. Consequently, the interlocking was lost in a short time and the surface got unravelled in a short time. So, it was considered that a matrix of soil round the aggregate particle was necessary to prevent rapid disintegration. Therefore, development of cementitious property of kankar was not taken into consideration.

The gradation analysis of soil-kankar aggregate mixtures compacted at the respective optimum densities was carried out. The results are given in Table 5.

TABLE 5
Mechanical Analysis of Kankar Before and After
Compaction

Limits specified by A. S. T. M.		Gradation of kankar before com- paction per cent passing	Gradation after compaction at optimum moisture and maximum density for different percentages of kankar. Percentage		
Sieve	Per cent Passing		20	40	60
1 in.	100	100	100	100	100
$\frac{3}{4}$ in.	70-100	85	90	86	87
$\frac{3}{8}$ in.	50- 80	65	66	66	67
No. 4 mesh	35- 65	50	52	52	51
No. 10 „	25- 50	35	40	38	35

The gradation of kankar after compaction was practically the same up to 60 per cent of kankar admixture.

Zeigler also pointed out that in standard proctor compaction test, the aggregate retained on 4 mesh is substituted by material passing No. 4 when this procedure is followed, it is necessary to apply correction because higher sized particles will increase the apparent density due to their higher specific gravity as compared with bulk specific gravity of the compacted dry soil. The correction often used is that the added coarse material displaces its volume of soil but has no further influence on the maximum density or other properties of the soil portion of the soil-kankar mixture and our conclusions are in conformity with those of Zeigler.

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KANKAR ON COMPACTION AND STABILITY OF SOIL

Maddison while studying⁶ the effect of the presence of stone content on compaction of cohesive soil had found out that at stone content higher than about 70 per cent, the stones were in contact with one another and so prevented any compaction of the soil mortar. Keeping this in view, higher percentages of kankar were not tried in the compaction test.

Correlation, if any, between stability value, C. B. R. and triaxial tests would be studied as suggested by Shri Narasimham, in due course of time. It had been included in the research programme.

EXTRACT FROM THE MINUTES OF THE MEETING OF
RESEARCH ORGANISATION DIVISION HELD AT NEW DELHI
ON THE 12TH DECEMBER 1956.

* * *

Shri Mehra explained briefly similar research and field experiments carried out by him a few years back in the Punjab, and said that the research work now being done was on the right lines and the conclusions were almost similar to those arrived at by him. After a brief discussion it was decided that the tests should be repeated on the basis of the suggestions made by Shri Narasimham and Shri Swaminathan who suggested that 'L' in McLeod formula given in Shri Narasimham's comments should be made equal to the unconfined compression strength of the mix. Another suggestion made by members was that C. B. R and Plate Bearing Tests should also be carried out.

* * *

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"ALKALI-AGGREGATE REACTION IN
CONCRETE CONSTRUCTION"

By

P. J. JAGUS* AND N. S. BAWA*

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SYNOPSIS

The principal sources of aggregate in some parts of the country are geologically identified to contain reactive constituents which are likely to cause damage to concrete if the aggregates from these sources are used in cement concrete construction. The problem has been studied with a view to safeguarding against the possibilities of alkali-aggregate reactions.

The different concepts and theories about this reaction put-forth by scientists at different times are analysed and discussed. The general belief that it is a safe to use low alkali cements with reactive aggregates is questioned through data collected in the Central Road Research Institute and also previously by others.

A general survey of the reactive aggregates in India is made. The alkali contents of the cements produced in different factories are given. Possible means of reducing the danger of alkali-aggregate reaction by the use of cheap inhibitors are described.

1. INTRODUCTION

1.1. Of the various possible deleterious interactions between aggregates and cements, the most important one is that between the alkalies in cement and certain chemically active minerals in the aggregates.

* Central Road Research Institute, New Delhi.

The principal damaging aspect of this reaction is the tendency of the product resulting from the reaction to swell excessively due to any one of the reasons put forth by the various researchers on the subject and cause distress and cracking in concrete. Failure of concrete would either be by tension caused by excessive swelling force from within or compression due to the prevention of expansion by restraint to the end movements. Tensile failures result in widespread crazing or cracking on the surface of the pavement if the reactive minerals are present in fine aggregates or random cracking and pop outs if such minerals are present in coarse aggregates. The rate of deterioration may be slow or fast depending upon the degree of reaction. The compressive failure may, however, occur suddenly as the restraint to end movement would give rise to an accumulation of stress which, beyond a certain point, might tend to crush or buckle the member.

1.2. Compared to foreign countries, particularly U.S.A. and Australia, where failures of concrete due to reactive aggregates have been quite extensive, the notable absence of any such failures in India may be a little surprising. One of the reasons for this may be the fact that the concrete construction is not so extensive as yet due primarily to shortage of cement and high construction cost. Where construction activity is high, only aggregates of proved performance are being used. In the wake of country's industrial development, construction activity will gradually penetrate deeper into those areas where it had not progressed so far. This would call for a larger use of aggregates from greater number of sources than at present.

1.3. Geologically India has very extensive deposits of volcanic rocks. The Deccan trap covering vast areas of Bombay and Madhya Pradesh, the limestones and dolomites of Madhya Bharat, Punjab and Rajasthan, traps and limestones from Jammu and Kashmir, would form extensive sources of aggregates for concrete construction.

1.4. The types of rocks which contain reactive constituents include traps, andesites, rhyolites, siliceous limestones and certain types of sand stones. The reactive constituents may be in the form of opals, cherts, chalcedony, volcanic glass, zeolites, etc. Their physical nature may be in the form of fillings of amygdaloidal cavities, veins, cleavages and nodules for cherts, opals, chalcydones, etc., or in a sub-microscopic or microcrystalline state for glasses. In the former state, identification of these minerals present in mother rock is possible by mere visual examination in most cases. It becomes, however, difficult to similarly identify fine grained igneous rocks like basalts, ande-

sites, rhyolites, etc., as reactive. Petrographic examination in such cases becomes necessary before an aggregate could be certified as innocuous or otherwise.

1.5. The main purpose of this Note is to assess the probable chances of alkali-aggregate reactivity and its ultimate effect on concrete construction, particularly in pavement construction. It also includes a general discussion on the possible use of inhibitors like pozzolanas, their different types and manufacture in India.

1.6. First Experiences

It was only in the late thirties, in the United States of America, that the detrimental effects of this reaction gave rise for the first time to failures of concrete pavements. The first reports were of a pavement in Salinas Valley, California ¹ which blew up early in 1938, only a year after its construction. Excessive expansion was the reported cause of this failure. Further reports of failures of other structures which included bridge piers, girders, sea walls and buildings in the same area followed. It was not till 1940 that Stanton ¹ was able to find out the real cause of these experiences and conclude definitely that the cracking was due to the interaction between the alkalis in cement and certain minerals in aggregates. The important observation of Stanton was that the expansion depended both on the alkali content of the cement and the amount of reactive minerals in the aggregates. For a cement of given alkali content, the expansion first increased with increasing amount of reactive material and then decreased. Based on these observations Stanton was able to put-forth his "Pessimum Ratio" concept.

1.7. After the publication of Stanton's original findings a number of other similarly affected concrete structures were studied in detail as to the sources of aggregates and alkali contents of cements used ². From these studies, an opinion that "the reaction in most cases was sequel to the using of high alkali cement" started gathering momentum.

1.8. Expansions and Alkali Content of Cements

Hanna in 1943, ³, however, showed as a result of his laboratory experiments on expansions of mortar bars using reactive aggregates and 132 different commercial cements, that "at an early age of 2 months these expansions, with only a few exceptions, were definitely related to the alkali content of the cements. At the age of six months, many exceptions were observed in this

relation and at the end of 18 months, nearly all the bars had expanded excessively. In many cases, bars of low alkali cement expanded to a greater extent than those of high alkali content."

1.9. Restriction on Permissible Alkali Content of Cements

Experiences of the field performances of structures in California, however, led to the strong belief that high alkali cements were the main cause of this deterioration and the maximum allowable alkali content of cements to be used in areas of reactive aggregates was restricted to 0.50 per cent Na_2O equivalent. This was the first restrictive step in specifying the cement quality in respect of its alkali content.

On the strength of serious deterioration at the Parker Dam⁴ within only 2 years after its construction, the Bureau of Reclamation, U.S.A. in 1940 imposed a limit of 0.60 per cent of alkali as Na_2O equivalent in the cements to be used for larger and more important constructions.

1.10. Deterioration of Concretes with Low Alkali Cements and Reactive Aggregates

Although the alkali content of cement was restricted below the specified values in areas of reactive aggregates, cases were not wanting where the deterioration of concrete made with such aggregates had taken place even with the use of low alkali cements⁵.

Donald Woolf in 1952⁶ after restudying the available data on laboratory tests using 35 different cements, also emphasized that the restriction on alkali content of cements cannot be a safe means of avoiding the dangers of this reaction.

1.11. Alkali: Opal Ratio

On the basis of these investigations, a different concept which laid emphasis on the ratio of the alkali content to the reactive mineral present in the combination of cement and aggregates emerged. Woolf defined this ratio as Alkali: Opal ratio.

According to this concept there exists a pessimum ratio for each type of reactive mineral which will give maximum expansion and deterioration. Alkali: Opal ratios higher or lower than the pessimum ratio will result in decrease in expansion.

In actual practice however, it would be impossible to define or apply this ratio because whilst the alkali content can be easily known, the reactive content of the aggregate equivalent to that of opal which would give an optimum value or pessimum ratio cannot be easily determined. This is on account of the fact that natural aggregates contain different types and forms of reactive minerals each having its own mineralogical index of reactivity. Also, the distribution of these minerals in the mass of concrete, and the surface area of these reactive particles together would affect this index of reactivity and in turn the pessimum ratio. If the dangers of the reaction are to be completely guarded against, the value of the alkali: opal ratio adopted in the field should be sufficiently different from the optimum.

1.12. "Safe and Unsafe Reactions." A Hypothesis

Very recently however, T. C. Powers and Steinour⁷ based on the laboratory data collected by Kalousek on the stabilities or otherwise of the alkali-silica complexes in presence of excess lime have put-forth a hypothesis on the 'safe and unsafe' alkali-aggregate reactions. According to this hypothesis, the normal alkali silica gel complex is of the swelling type. If, however, calcium available as free lime can dissolve in the water, it can combine in varying degrees with the alkali silica gel to give calcium-alkali-silica complexes which are of a non-swelling type. Solubility of calcium ions in water depends upon alkali concentration of the water. If the alkalinity is high, calcium does not go into solution and cannot therefore penetrate to the reaction spots where alkali gels of the swelling type are getting formed. This is termed as "unsafe reaction". The incidence of this reaction is higher where high alkali cements are used because the available water soluble alkali is very much more in such cases.

1.13. Kalousek was able to confirm by laboratory experiments that with alkali concentration of 0.2 gm of Na_2O per litre the composition of the gel was $0.003 Na_2O : 2.0 CaO : 1.0 SiO_2 : xH_2O$. But when alkali concentration was raised to 20 gm per litre and above, the composition of the gel was $0.25 Na_2O : 1.0 CaO : 1.0 SiO_2 : xH_2O$. Such higher molar concentration of Na_2O will result in imparting swelling characteristics to the gel.

Table 1 gives the alkali contents of cement which would give rise to alkali concentration of 20 gm per litre of the free water for water-cement ratios of 0.50, 0.60 and 0.70 which are normally adopted in India.

TABLE 1

**Alkali Content of Cement required for Alkali
Concentration of 20 gm per litre of free water**

i Wreat gm per gm of cement	Required alkali content of cement			
	0.10 gm combined water* per gm cement	0.15 gm combined water per gm cement	0.20 gm combined water per gm cement	0.25 gm combined water per gm cement
0.50	0.80	0.70	0.60	0.50
0.60	1.00	0.90	0.80	0.70
0.70	1.20	1.10	1.00	0.90

It is evident that even with medium alkali cements, there is every likelihood of reaching the alkali concentration of 20 gm per litre and thus resulting in unsafe reaction.

1.14. Confirmation of the Significance of Pessimum Proportions

Laboratory tests to verify the effects of the relative contents of alkali and reactive aggregates on expansion of concrete were made. Pyrex glass was used as a standard reactive aggregate and alkali content was varied by addition of $NaOH$. Cement used in these tests had an alkali content of 0.55 per cent Na_2O equivalent.

Mortar bars were cast using graded pyrex glass as aggregate and cement with percentages of alkali varying from 0.55 to 2 per cent. Enough water was used to provide a flow of about 100 to 115 per cent. The specimens were stored in sealed containers at 122° F. and 100 per cent humidity. Changes in length of specimen were observed after 14 days. Fig. 1 shows the percentages of expansions of bars for varying alkali contents. It is of interest to note that the expansions are of a high order when the alkali content approaches 1.23 per cent. For alkali values higher or lower than this, the expansions are less than that for 1.23 per cent. The minimum expansions observed for low alkali

* The values of combined water varying from 0.10 to 0.25 gm per gm of cement are based on data given by Powers and Steinour⁷ for neat cement pastes over two years old using two water-cement ratios of 0.45 and 0.65.

cements are low, but those for alkali contents greater than 1.23 per cent, though less than the maximum, are not very low. This may possibly be due to the fact that, for the formation of alkali silica combinations in this range, some gels satisfying the pessimum proportion may also be formed simultaneously as the alkali available is in excess and thus result in some expansion.

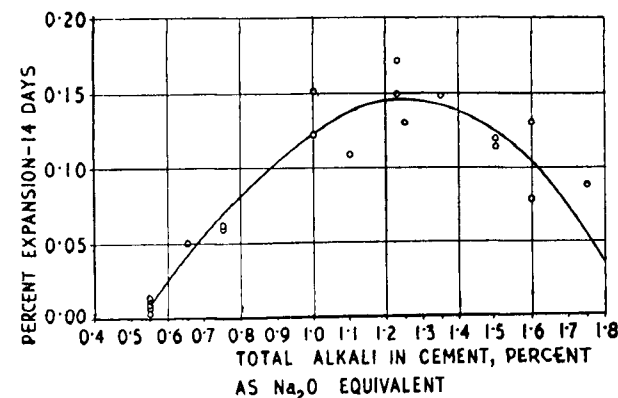


Fig. 1. Effect of alkali content of cement on mortar-bar expansion using graded Pyrex glass

Another series of tests was run in which instead of varying the alkali content, it was kept constant and the percentage of reactive aggregate (pyrex glass) was varied. The inert aggregate used in these tests was graded Ennore sand. Fig. 2 shows the expansions of mortar bars using fixed alkali content of 0.55 per cent and varying percentages of reactive aggregate mixed with innocuous aggregate. Here, too, the curve displays characteristics similar to those of Fig. 1. Maximum expansion occurs when the reactive material is 30 to 40 per cent. Percentages lesser than or in excess of this optimum cause expansions less than the maximum.

It is possible to surmise from the above that with the combinations of those respective percentages of alkali and reactive aggregate giving the optimum proportions, the gel complexes formed swell in presence of water. Complexes containing excess of alkali are in a liquefied state and those containing excess silica are non-swelling type. Expansions in both the cases, therefore, are of lesser order. Also, medium or less reactive aggregates can be more dangerous with medium or low alkali cements, than with high alkali cements.

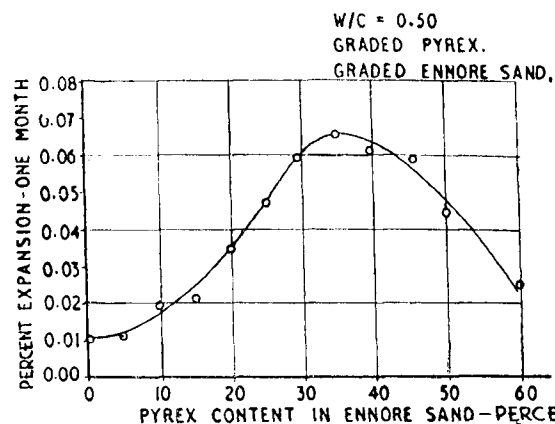


Fig. 2. Effect of varying percentages of reactive material (Pyrex glass) in Ennore sand on mortar bar expansion

Further laboratory tests have been undertaken and field tests are being planned to confirm the above findings using opal and natural reactive aggregates.

1.15. Mechanism of Reaction and Swelling

Stanton's investigations¹ and subsequent researches by various workers established that the cause of expansion and cracking was the result of the formation of alkali silicates due to combination of alkalis in cement with reactive silica in the aggregates. The alkali silicates are in the form of very viscous gel which absorbs water and thus create tremendous swelling pressure.

Hansen in 1944⁸ put forward his osmotic pressure theory. The osmotic pressure was exerted by the alkali silicate solution enclosed in semi-permeable membrane. It was suggested that hardened cement paste around the aggregate particles acted as the semi-permeable membrane which allowed alkalis to diffuse through it to react with the reactive particles, but prevented the complex alkali silicate ions from passing out.

Vivian⁹ (1947) rejected the osmotic pressure theory and put forward his gel theory. He said that the viscous silicate gel could exert pressure due to its very viscous nature and rigidity. A semi-permeable membrane was not necessary to develop the pressure. It is believed that forces producing expansion were present only when the gel was sufficiently rigid, but as soon as water sufficient to make the gel sufficiently fluid was imbibed, it flowed through the voids and released the swelling pressure.

General opinion is shifting towards Vivian's theory which can explain such phenomenon as reduction in expansion with more than optimum water, high temperature and with increase in alkali content in the mortar after a certain limit given by Pessimum Ratio.

Vivian¹⁰ was able to prove that alkalis have the property of liquefying the silicate gels.

1.16. The two hypotheses can, in fact, be reconciled by attributing the initial expansion to the swelling pressure by gel formation. The gel, on liquefaction escapes to the surface or into the adjoining voids and cracks which in turn gives rise to further expansion due to osmotic pressure. It is recognised that the initial expansion due to swelling pressure exerted by the gel is the important one and is the cause of most of the expansion and cracking.

2. REACTIVE AGGREGATES

2.1. Types of Reactive Aggregates

Although, the earlier investigations showed that opaline silica is the most likely source of reactive aggregates, subsequent works have revealed that various minerals in rocks could cause trouble but in varying degrees. Appendix I gives the classification of these minerals based on their geological formation, degree of reactivity, etc., and modes of identification.

2.2. Methods of Testing the Reactivity of Aggregates

Different tests have been evolved by various researchers but each has its own advantage as well as limitation.

(a) Petrographic Examination

This is a very reliable test for identifying the presence of reactive constituents with microscopic examination of thin rock sections. A quantitative estimate of the minerals present is also possible. This test is always supplemented by other tests.

(b) Chemical Test

A 25 gm of crushed (52 - 100) sample is immersed under 25 millimoles of one normal NaOH solution in a stainless steel container for 24 hours at 80° C. The solution is analysed for dissolved silica and free alkali present. If the ratio of dissolved silica to reduction in alkalinity both expressed as millimoles per litre exceeds one, the aggregate can be reactive.

A good co-relation is observed between this and the mortar bar test given below.

(c) Mortar Bar Expansion

This is a direct test initiated by Stanton.

Aggregate of the following gradation is used :

Per cent	Passing sieve	Retained on sieve of mesh
19	$\frac{3}{16}$ in.	7
19	7	14
19	14	25
19	25	52
5	100	—

Specimens of size 1 in. by 1 in. by 10 in. are cast using enough water to give a flow of 100 to 115 per cent. After 24 hours of humid curing, the bars are measured and stored in sealed containers kept at 122° F. and 100 per cent humidity.

Lengths of the bar are observed at regular intervals. Expansion of 0.040 per cent at the age of six months and 0.1 per cent at one year are considered serious.

(d) Tensile Briquette Tests

This test was suggested by Vivian¹¹ believing that with the swelling pressures developed inside the mortar using reactive aggregates, its tensile strength would be adversely affected. If the gel formation takes place at early ages, swelling pressures would develop and will be indicated by loss in tensile strength.

2.3. Discussion on Methods of Testing

Of the above mentioned tests the most reliable ones are the petrographic examination of the aggregates and the mortar bar tests. The latter test is time consuming. So, it may not be directly applied for investigating the suitability of aggregates for specific jobs unless the tests are done well in advance of starting the work. However, it would prove very useful for testing aggregates from established sources.

The chemical test for finding out the dissolved silica and reduction in alkalinity does find favour as a confirmatory test to petrographic and mortar bar tests. The ratio of $\frac{Si}{Ra}$ of 1.0 laid down by Meilenz¹² and others was recommended after testing aggregates of known performances and co-relating the test values with their field performances.

The tensile briquette test may provide an easy means of early identification of the presence of reactive minerals in the aggregates. The drawback of the test is that the tested section at the necking of the briquette may not always be the section where the reaction has actively progressed and weakened it. This factor can partly be overcome by using a large number of test specimens in each series.

2.4. Indian Aggregates

To make a general survey and make a statement as to whether the aggregates from certain parts of the country are safe or unsafe would be considered too unwise looking to the vast expanse of the country and the wide manner in which the deposits from one locality can differ from another in spite of their being in the neighbourhood of each other. By examining the basic rock types of the country which exist in extensive deposits covering very large areas, a fairly comprehensive picture could be drawn up to enable one to conclude as to whether these rocks could contain constituents which might be significant from point of view of alkali-aggregate reaction.

It can be hoped that in quite a few parts of north, east and south India, aggregates could be non-reactive as their mode of geological formation is not conducive to the easy formation of reactive minerals. This can, however, be a pure supposition in the case of some sources of aggregates which may be of the nature of local deposits and may contain harmful minerals.

The principal common types of rock aggregates of India, considered suitable for use in concrete, are :

- (1) granitic, coarse grained, igneous rocks ;
- (2) basaltic, fine grained, igneous rocks ;
- (3) limestones ; and
- (4) sandstones or grit stones and quartzites.

2.5. Granite and Granitic Rocks

These would include coarse-grained igneous rocks which have crystallised on account of their slow rate of cooling during formation stage. Gabros, gneisses, diorites and granodiorites are also included in the group, but the grains are elongated or better interlocked. Gabros, gneisses and diorites are not alkali reactive but granites, pigmetites could be reactive by the presence of rhyolites and glass. Weathering can also produce

opal which is highly reactive. These occur mostly in South India around Madras, Bangalore and Seringapatam.

2.6. Basalts, Diabase and Dolerites

These rocks are fine grained and range from glassy to micro-crystalline. Sudden cooling during their formation creates a glass phase which may be more acidic or less acidic than the crystalline phase. More acidic glasses in basalts are definitely alkali reactive whereas the coarser matrix of the diabbases is non-reactive. Regions of occurrence are the Deccan plateau, West Coast, Bombay State, Madhya Pradesh, Kathiawar, Madhya Bharat, Hyderabad, Panjal Hills (Jammu & Kashmir), Bengal and Bihar.

2.7. Limestones

These include dolomites. Basically innocuous, limestones or dolomites can contain chert nodules which would be highly reactive. Indian limestones of Bijawar series are known to be highly cherty. Regions of occurrence include Madhya Bharat, Rajasthan, Punjab and Assam.

2.8. Sand Stones and Quartzites

Sand stones are generally safe for use in concrete if their physical characteristics are up to the required standard. These, however, sometimes contain highly reactive secondary silica minerals as binders or otherwise and may cause expansion and failure of the aggregate itself. Secondary silica minerals are chalcedony, crypto to microcrystalline quartz or opal.

Just as sand stones, quartzites can also occasionally be reactive. The interstitial quartz in quartzite deposited in crystallographic continuity with the grains is not reactive. In certain cases only, this binding matrix is reactive. Regions of occurrence include Madhya Pradesh, Madhya Bharat, Bengal, Bihar and Delhi.

2.9. From the survey given in Para. 2.4 to 2.8 it can be seen that there are fair chances of coming across reactive aggregates in construction works. Deccan traps covering vast areas nearly 2,00,000 square miles call for a very careful and detailed survey of their quality from place to place. From the nine samples of principal types of road aggregates tested, the basaltic samples received from Madhya Pradesh, West Bengal and Kashmir were found by mere visual examination to contain substantial quantities of reactive minerals like opals, chalcedony



Photo 1.
Kashmir trap (Banihal) showing white reactive silica.



Photo 2. Deccan trap (Nagpur) showing cavity fillings and veins of reactive silica.

and amorphous silica, Photos 1 and 2. Quartzite samples received from Kashmir were also found to be highly reactive. Out of 17 samples received four were identified to be reactive by visual examination.

2.10. Tests on Reactive Aggregates

Geologically, the three aggregates tested namely, limestone, trap, and quartzite were different. Of these, the trap aggregate was the only one which appeared to the naked eye to contain reactive constituents. Only negligible expansion took place with limestone, Fig. 3. The expansions of the mortar bars using

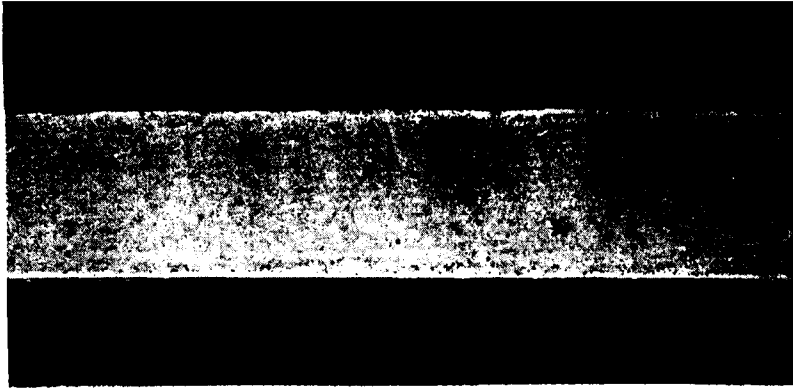


Photo 3. Non-reactive limestone mortar bar free from reaction spots.



Photo 4. Reactive quartzite mortar bar showing reaction spots in wet condition.

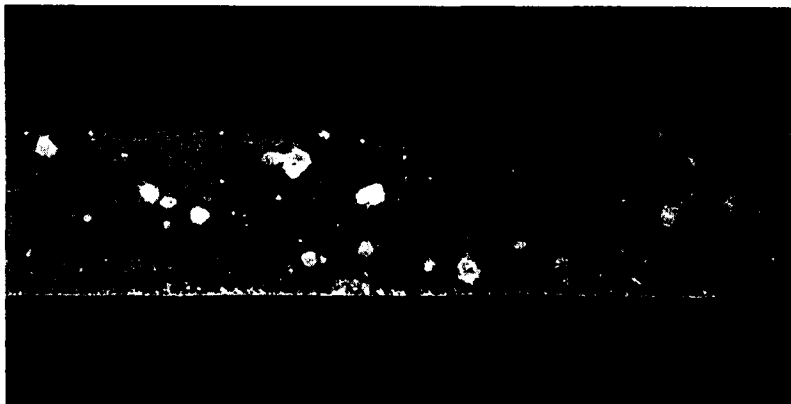


Photo 5. Reactive quartzite mortar bar showing reaction spots dried up.

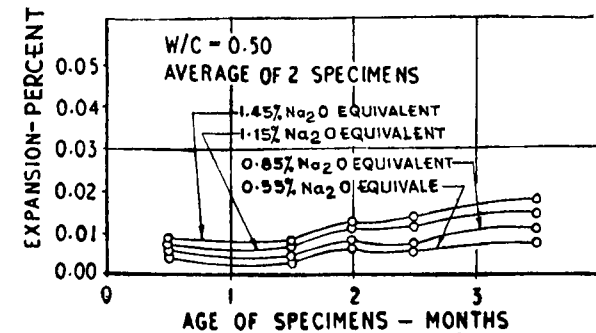


Fig. 3. Mortar bar expansion of innocuous limestone (J & K) with different alkali contents of cement

different percentages of alkali distinctly reveal reactivity in the case of trap as shown in Fig. 4. The specimen using

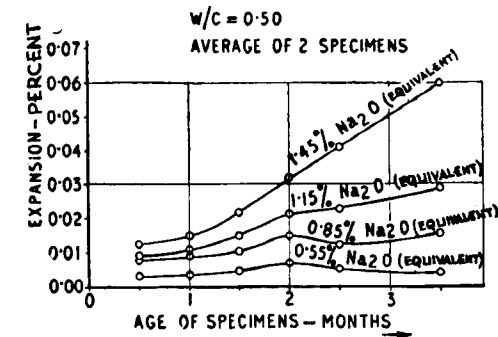


Fig. 4. Mortar bar expansion of reactive trap (J & K) with different alkali contents of cement

quartzite expanded most, Fig. 5. Reactive spots shown in Photos 4 and 5 developed on the faces of quartzite mortar bars, indicating presence of active silica in both the rock types.

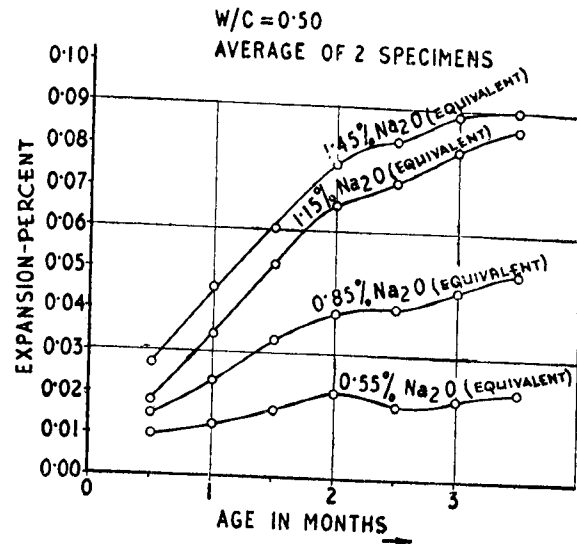


Fig. 5. Mortar bar expansion of reactive quartzite (J & K) with different alkali contents of cement

2.11. Reduction in Alkali and Soluble Silica Determinations

Chemical tests were conducted on the samples and the values of ' R_a ' and ' S_i ' determined. The results are given in Table 2. The ratios of $S_i : R_a$ greater than unity, confirm the finding of the mortar bar test on quartzite and trap. Total absence of any soluble silica in limestone confirms its innocuousness as given by mortar bar tests.

Results of similar tests carried out on samples of other road aggregates suspected to be reactive on visual examination are also given in Table 2. Although the ratio of $\frac{S_i}{R_a}$ is less than unity in cases of Rajmahal and Deccan traps, the presence of very large quantities of reactive silica is indicated by very high values of S_i .

The high activity of pyrex glass used as a standard reactive material for laboratory tests is also indicated by this test.

TABLE 2

Dissolved Silica and Reduction in Alkalinity of some Indian Aggregates

S.No.	Description of Aggregate	Reduction in Alkalinity millimoles per litre R_a	Dissolved Silica millimoles per litre S_i	Ratio $\frac{S_i}{R_a}$
1.	Banihal Trap	16	42	2.63
2.	Deccan Trap (Nagpur)	320	177	0.55
3.	Rajmahal Trap (West Bengal)	252	210	0.83
4.	Banihal Quartzite	16	57	3.56
5.	Limestone	24	Nil	Nil
6.	Limestone (Balasore)	96	Nil	Nil
7.	Limestone (Banswara)	32	Nil	Nil
8.	Ennore Sand	4	Nil	Nil
9.	Gravelly Sand No. 1 (Banihal)	68	Nil	Nil
10.	Gravelly Sand No. 2 (Banihal)	100	Nil	Nil
11.	Pyrex Glass	160	926	5.79
12.	Opal	120	721	6.01

2.12. Tensile Briquette Tests

Since the presence of reactive constituents in the trap and quartzite samples has been proved, tensile briquette tests using these aggregates were made to see if this test, which is not accepted as a means of finding alkali-aggregate reactivity so far, can be used to any advantage. Briquettes with mortars having different alkali contents were cast using graded crushed rock screenings. Specimens were prepared for testing at 15 days and one, three and six months. Whilst writing this Paper, only results

for 15 days and one month were available and are considered sufficient to hold interest in the study. Briquettes of Ennore sand were also included in these tests as it was being used in laboratory tests as innocuous material. The results are given in Table 3 and Fig. 6.

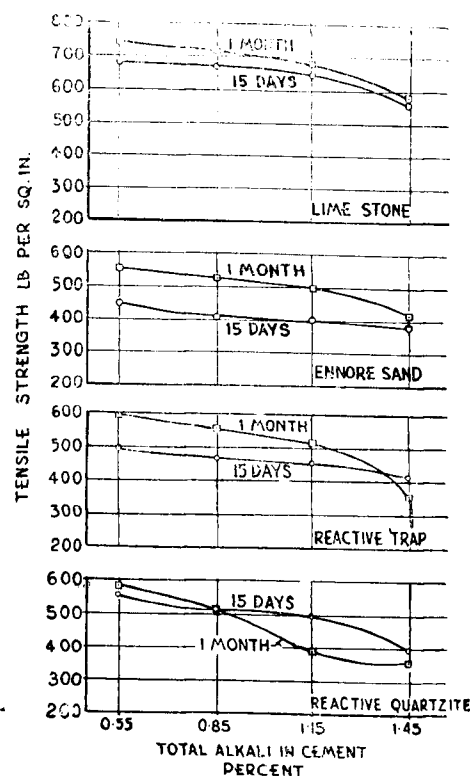


Fig. 6. Tensile strength of briquettes with different aggregates and cements of different alkali contents

The increasing alkali content tends to reduce the tensile strength of briquettes in all cases. The rate of gain or strength of briquettes with age is also reduced with increasing alkali content. With reactive aggregates this rate of gain of strength with age became negative, i.e., lower strengths were obtained at greater ages. More reactive the aggregate the more easily they will lose strength with increasing alkali content and age.

TABLE 3

Tensile Strength of Mortar Briquettes using Graded Aggregates and Cements of different Alkali Contents

Total alkali content of cement per cent	Tensile Strength in lb per sq. in.							
	Reactive Quartzite		Reactive Trap		Ennore Sand		Limestone	
	15 days	30 days	15 days	30 days	15 days	30 days	15 days	30 days
0.55	550	573	491	597	449	553	674	738
0.85	513	511	468	558	406	522	663	711
1.15	497	386	456	514	394	492	640	670
1.45	394	357	416	359	370	411	553	576

The above interesting observations indicate the possibility of adopting the briquette test for identification purposes particularly because it can serve as a very rapid means of identifying the presence of deleterious minerals. Further work on this test is in hand.

3. INDIAN CEMENTS

3.1. Effect of Alkali Content

Precautionary measures are called for wherever a reactive aggregate is likely to be used either with high or low alkali cements. The nature and degree of such precautionary steps would naturally vary with the alkali content of cement in each case.

3.2. Alkali Determination of Indian Cements

No data are available on the alkali content of Indian portland cements. This is due firstly, to the fact that, no importance has so far been given in India to the alkali-aggregate reaction in concrete and secondly, it is difficult to determine accurately the alkali content. In order to find out the alkali contents of Indian cements K_2O and Na_2O determination were done by Perkin Elmer Flame Photometer on samples of cements

received from 26 factories in sealed containers. The method of analysis followed was ASTM Standard, C 228-49T.

Fig. 7 gives K_2O , Na_2O and total alkali content as Na_2O equivalent of the 26 different brands. The analysis has been done only on one sample from each factory and there is a possibility of slight fluctuations in day to day manufacture in each case. However, the data is very useful for forming a general idea about the levels of alkali contents of Indian cements. It is evident from Fig. 7 that:

- (i) K_2O contents of cements are generally higher than the Na_2O contents.
- (ii) The minimum and maximum total alkali contents are 0.15 and 0.99 per cent respectively.
- (iii) The analysis given below shows that eleven out of 26 cements have total alkali contents higher than 0.60 per cent, which is the arbitrary safe limit fixed in American practice.

Alkali content per cent	No. of Samples	Percentage of Number of samples to the total
Below 0.40	8	30.8
0.40-0.60	7	26.9
0.60-0.80	5	19.2
0.80-1.00	6	23.1
Above 1.00	Nil	—

- (iv) The maximum alkali contents are not as high as those come across in the U. S. A. and the U. K. where maximum percentages of more than 1.0 per cent are known to have been reached. All the same, in the light of the discussion mentioned above the problem with the above range of alkalis has an equally great importance in India.

4. IMPORTANCE OF ALKALI-AGGREGATE REACTION IN CONCRETE PAVEMENTS

4.1. Experiments on relative performance of concrete using reactive aggregates have proved beyond doubt that the deleterious action is far more pronounced in concretes exposed to climatic

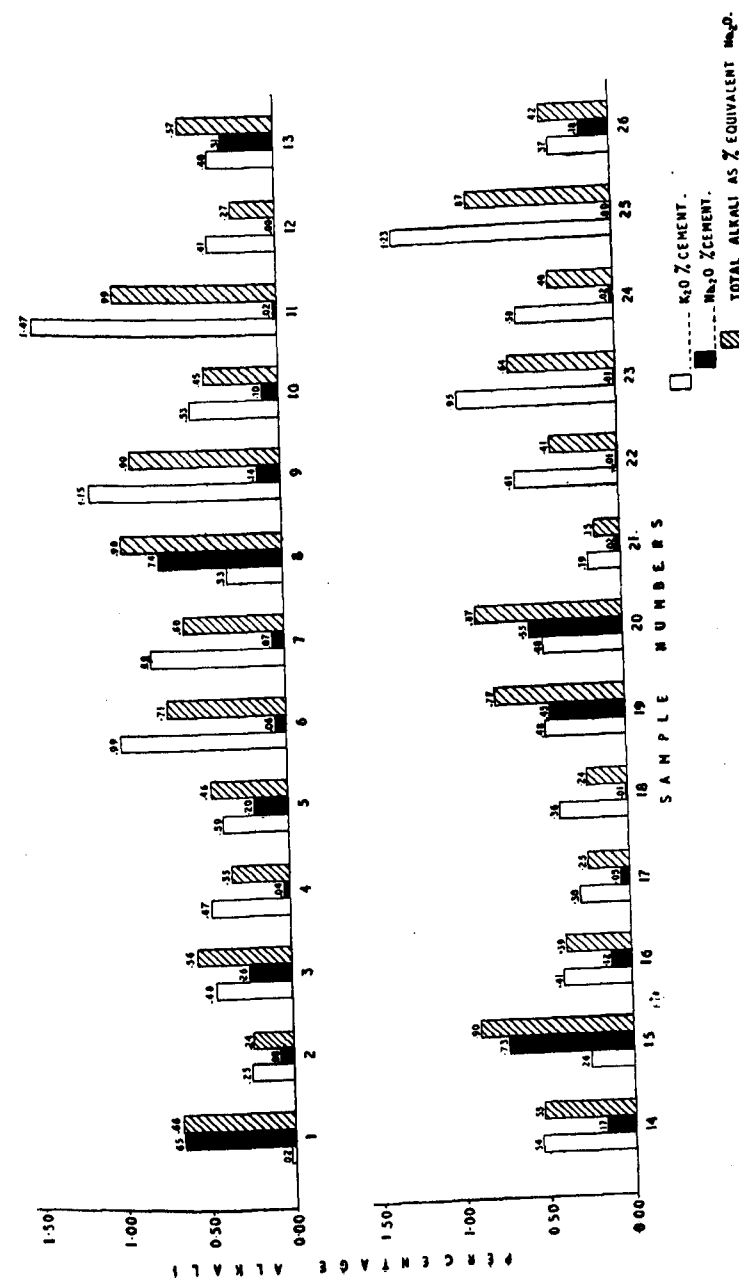


Fig. 7. Alkali contents of Indian cements

changes like rise and fall in temperatures, wetting and drying, etc., than when the concrete is in sheltered situations. The wetting and drying of the mass can adversely affect the concentration of the alkalinity and its mobility in the pore structure which may tend to aggravate the reaction with the help of temperature changes. This fact is very important in concrete pavements where the area of concrete exposed to weathering per unit volume is far greater than in other outdoor structures. Also, the bottom face of concrete being in intimate contact with the subgrade may undergo changes in temperature and moisture parallel to it. In case the soil is alkaline, there is a danger of additional alkali being available for reaction from the soil.

4.2. Structurally, the slenderness of the pavement being high, its resistance to the internal distresses caused by the reaction is naturally lower than in other comparatively massive structures. The effects of the internal stresses developed become more easily apparent in pavements as the cyclic changes in temperature and humidity conditions, combined with wheel load effects, lead to frequent reversal of stresses. Greater attention therefore, is warranted in the selection of aggregates for use in pavements and other constructions where the sections are thin and therefore get an easy coincidence with outside climatic changes.

5. PREVENTIVE MEASURES

5.1. All means and methods which could either eliminate, or convert into harmless state those compounds which contribute to alkali-aggregate reaction and development of swelling pressure deserve acceptance provided they do not give rise to any secondary detrimental effects. The following are some of the proved methods of eliminating or reducing the potential dangers of expansion when reactive aggregates are used:

- (a) Using low alkali cements
- (b) Using air entrainment
- (c) Waterproofing treatment
- (d) Mixing pozzolanic admixtures to concrete

5.2. Use of Low Alkali Cements

Though a limit of 0.60 per cent of Na_2O equivalent on alkali content of cement has been set in American practice for all types of concrete construction, recent researches have established the possibility of delayed expansions with low alkali cements

containing as low as 0.2 per cent of Na_2O equivalent, when the percentage of reactive minerals in aggregate is also proportionately small. In the light of these findings and considering the difficulties in the manufacture of very low alkali cements such as procuring of selective raw materials, high temperatures of burning etc., not much reliance can be placed on this method.

5.3. Use of Air Entrainment

Use of air entrained concrete to prevent expansion is based on release of swelling pressure due to escape of the gel into the air voids in the body of concrete. McCoy and Caldwell¹³ have shown that high reductions in mortar bar expansions occur when air-entraining admixtures are used. However, while adopting this method for prevention of deleterious expansion the possibility of reduction in the strength of concrete as a result of air entrainment should be kept in mind. With high alkali cements and highly reactive aggregates this may give only a temporary relief as swelling pressures would develop when the quantity of gel formed requires more void space than that provided by air entrainment.

5.4. Waterproofing Treatment

Presence of water is essential for the silicate gel to exert swelling pressure. Prevention of access of outside water into the body of concrete by waterproofing or other means may afford partial relief. As water is always present in mixed concrete and the quantity added is normally sufficient to permit the reaction taking place, preventive measures such as this would help only to reduce the severity of the reaction.

5.5. Use of Pozzolana and Pozzolanic Action

When portland cement in concrete hydrates, large quantities of calcium hydroxide (hydrated lime) are released. This free lime does not contribute towards the strength of concrete. On the other hand, its presence imparts many undesirable properties to concrete such as susceptibility to leaching by water, attack by sulphates and efflorescence. A pozzolana, while in itself possessing no cementitious properties, reacts in the presence of moisture with the liberated calcium hydroxide to form compounds possessing cementitious properties. Also the fixation of free lime will make the concrete more resistant to leaching by water and sulphate.

The action of pozzolanas as inhibitors of the alkali-aggregate reaction is a bit different. Inhibition of the reaction is made

possible in two different ways depending upon the type of pozzolana employed :

- (i) Fixation of the alkali of cement by providing highly reactive material similar to or other than the reactive silica present in the aggregate.
- (ii) Removal of alkali ions from solutions by absorption or chemical combinations giving rise to insoluble complexes.

Pozzolanas which inhibit the alkali-aggregate reaction in accordance with (i) mentioned above can be further subdivided depending on the nature of the reactive constituent. Opal, calcined opaline clays and shales, volcanic glass like pumice etc., have silica as the reactive constituents and are therefore called siliceous pozzolanas. On the other hand materials like fly ash contain reactive alumina which forms aluminates with alkalies in the cement and are, therefore, "non-siliceous".

When siliceous pozzolanas are used, the fixation of the alkali by the silica in the pozzolanas gives rise to the formation of secondary alkali-silica complexes similar to the ones formed with reactive aggregates. So, unless pozzolanas are used in large quantities, they do not become effective. With high alkali cements and moderately or slightly reactive aggregates, these pozzolanas used in quantities less than the required would give rise to even greater expansions than when they were not used. In the case of fly ash, however, there is no formation of secondary silicate gel and danger of expansion with small additions of pozzolana is absent.

Pozzolanas which act in accordance with (ii) mentioned above include certain clays of the montmorillonite and kaolinite type, calcic zeolites, etc. These pozzolanas reduce the concentration of alkali and hence the solubility of reactive silica. Therefore, their effectiveness increases with the addition of increasing percentages.

5.6. Suitability of Pozzolanas

Numerous tests have been suggested for verifying the efficacies of pozzolanas. These are briefly discussed below :

(a) Lime Reactivity Test

According to Dr. Lea's definition, a pozzolana combines with free lime to form insoluble calcium silicates and aluminates, which are cementitious and impart strength giving proper-

ties. This property is used in this test. Specimens are cast using lime, pozzolana and sand in the proportion of 1 : 2 : 9 and tested for compressive strength after 7 days of curing at 50° C and 100 per cent humidity.

Classification of the pozzolana is done in accordance with the following :

Compressive strength in lb per sq. in.	Activity index
Below 200	Very inactive
200 - 400	Inactive
400 - 600	Poorly active
600 - 800	Medium active
800 - 1000	Active
Above 1000	Very active

Lime reactivity test does not, however, provide information on the inhibiting qualities of the pozzolana in alkali aggregate reaction and is not very useful from this standpoint.

(b) Chemical Composition

Although it is generally agreed that silica, alumina and, as recently found, iron content are the principal ingredients of active pozzolanas, no co-relation has been reported between the chemical composition and reactivity. This test should, therefore, be used only for mandatory purposes such as limiting or prohibiting the presence of harmful secondary ingredients like SO_3 , MgO , carbon, etc.

(c) Mortar Bar Expansion Test

This is a direct method of measuring the corrective power of a pozzolana for alkali-aggregate reactivity. In the laboratory, expansion is obtained using pyrex glass as a reactive aggregate and high alkali cement. The percentage reduction in expansion of 1 in. by 1 in. by 10 in. mortar bars with different percentages of pozzolana is used as the basis of its reactivity.

Contrasted to its application in aggregate testing, the mortar bar test is a much more quicker and surer means of testing a pozzolana.

(d) Reduction in Alkalinity and Dissolved Silica Test

In this test 12.5 gm of pozzolana are treated with 25 gm of one normal $NaOH$ solution for 24 hours at 80° C. and the reduc-

tion in alkalinity of the solution 'Ra' and the quantity of dissolved silica 'Si' are determined.

Good siliceous pozzolanas would give high 'Si' values whereas other good types would provide higher 'Ra' values. The pozzolanic quality could be judged from the following average values expected of good pozzolanas.

Ra	Si	Type
Slightly and	400—700	Opal and opaline pozzolanas
Above 300 and	50—300	Calcined clays
200—400 and	150—400	Aluminous silica glasses like pumicities.

Contrasted to lime reactivity test, this test is significant only for alkali-aggregate reactivity.

5.7. Surkhi as Pozzolanas

An intensive research study on the use of surkhi as a pozzolana in concrete construction is being conducted at the Central Road Research Institute for the last three years. The findings¹⁴ are very important as they clearly point out to certain misconceptions on the type and suitability of the surkhi as commonly used. Some of the very important conclusions of the study are:

- (i) Soils suitable for brick making do not make good surkhis. Clayey soils give generally better surkhis.
- (ii) Each type of clay or soil has a definite temperature burning which gives maximum reactivity. The reactivity gets reduced if temperatures higher or lower than the optimum are adopted. The terms 'underburnt', 'wellburnt' and 'overburnt' normally used for surkhis are technically incorrect.
- (iii) Kaolinite and Montmorillonite types of clays can make good surkhis when burnt in 600°-800° range. Illitic clays require higher temperatures of burning and may prove uneconomical for use.
- (iv) Red or yellow soils with free iron oxide generally yield better surkhis than black cotton soils.

The above study dealt mainly with those aspects of surkhis which affected their combination with free lime. Lime reactivity test was the only basis for finding the indices of their activities.

5.8. Surkhi as inhibitor of Alkali-Aggregate Reaction

Table 4 gives the values of Ra and Si and lime reactivity on some clays calcined at different temperatures. The importance of correct temperature of burning is very apparent from the wide changes in values of Ra, Si and compressive strengths. There appears to be no co-relation at all between Ra and Si values and the lime reactivity values.

TABLE 4
Dissolved Silica, Lime Reactivity Values, and
Reduction in Alkalinity of Some Surkhis (Accelerated Test)

S. No.	Description of Surkhi and tempe- rature ef burning	Reduction in Alkali- nity (milli- moles/litre) <i>Ra</i>	Dissolved Silica (millimoles/ litre) <i>Si</i>	Compressive Strength (Lime Reacti- vity Test) lb per sq. in.
1.	Rayarkeri Clay 400°C	512	Nil	453
	" " 600°C	600	13	1512
	" " 800°C	424	419	1444
	" " 1000°C	244	407	822
2.	White Kaoline (commercial) 600°C	264	Nil	1385
	" " 1000°C	164	513	889
3.	Jamna Bank Clay 400°C	240	Nil	133
	" " " 600°C	236	Nil	324
	" " " 800°C	232	42	839
	" " " 1000°C	326	435	1133
4.	Delhi Kaolin I 600°C	288	2	1837
	" " 800°C	220	3	1215
5.	Delhi Kaolin II 600°C	336	6	2015
	" " 800°C	220	Nil	1896
6.	Kashmir Clay 600°C	256	20	317
	" " 800°C	200	103	600
	" " 1000°C	72	240	385
7.	Yellow Ochre 800°C	360	Nil	1837
8.	Red Ochre 800°C	328	Nil	1496

The types of minerals involved in pozzolanic reactions being more than one their modes of activity also vary from one to another. At different temperatures, the same material shows high activity with reference to one test and very low activity for another. In some cases the activities indicated by both the tests are high.

For judging its effectiveness against alkali-aggregate reactivity, the R_a and S_i values from Table 4 appear to be at least of some use than those of the lime reactivity test values.

The materials mentioned in Table 4, when used as inhibitors gave varying percentages of reduction in the expansion in mortar bar tests. Both Rayarkeri clays and kaolin clays show themselves as strong inhibitors of alkali-aggregate reactivity. Fig. 8 and 9 show the effectiveness against alkali-aggregate reaction of different percentages of addition of the Rayarkeri clay burnt at 600° C. From the comparison of test values given in Table 4, it can be seen that this property cannot be judged from the R_a and S_i values alone.

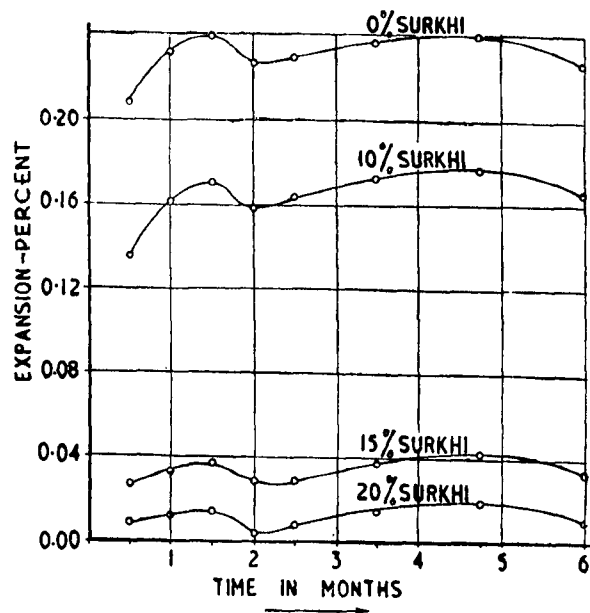


Fig. 8. Effect of addition of Rayarkeri surkhi on mortar bar expansion using graded pyrex and high alkali cement

Surkhis made by calcining to 600 and 800 degree C. clay obtained from four different sources, have been tested for inhibition of alkali-aggregate reactivity by the mortar bar expansion test. The results are given in Table 5 on Page 70.

Data on tensile briquette test which has shown some promise in testing the reactivity of aggregates are also being collected.

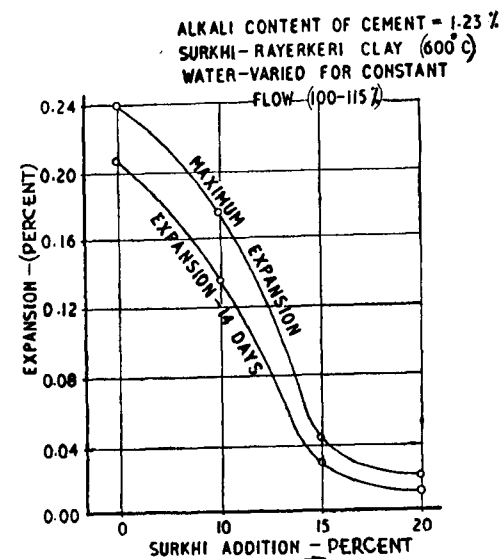


Fig. 9. Effect of addition of Rayarkeri surkhi on mortar bar expansion using graded pyrex and high alkali cement

CONCLUSIONS

6.1. India has numerous sources of potentially reactive aggregates.

6.2. The alkali contents of Indian cements vary fairly widely although the maximum is comparatively lower than the highest values obtained in foreign countries.

6.3. With a range of alkali contents and water-cement ratios normally adopted in India, the molar concentration of alkali can be sufficiently high to give rise to unsafe reactions with reactive aggregates.

TABLE 5
Reduction in Expansion of Pyrex Glass Mortar Bars
Using Different Surkhis

S.No.	Surkhi Description.	Temperature of burning.	Per cent Addition.	Reduction in Expansion. Per cent	
				14 days	1½ months.
1.	Rayerkeri clay	600°C	10	35	29
			15	86	85
			20	96	94
2.	Delhi Kaolin No. 1.	600°C	15	100	100
			20	100	97
		800°C	15	86	86
			20	92	95
3.	Delhi Kaolin No. 2.	600°C	10	89	87
			15	95	93
			20	95	93
		800°C	15	97	98
			20	92	93
4.	Kashmir clay	600°C	15	48	43
			20	-37	-43
		800°C	15	59	49
			20	69	57

6.4. Wherever reactive aggregates have to be used, the adoption of pozzolanic cements or pozzolanic admixtures would prove a more reliable measure of safeguard than going in for a low alkali cement.

6.5. It is necessary to develop extensively the manufacture and use of pozzolanas and pozzolanic cements as a means of safeguard against deleterious reactions. This would also result in an indirect economy in construction costs.

6.6. If a pozzolana is to be used as an inhibitor of alkali-aggregate reaction, its efficacy should always be tested by the mortar bar expansion test in addition to other tests.

6.7. The tensile briquette test holds promise as a quick test for judging the potential reactivity of aggregates.

6.8. The detrimental effects of alkali aggregate reactivity, if any, would tend to show up much faster in concrete pavements than in other types of exposed structures because of the comparatively thin unreinforced sections that are generally used in India. The subject is, therefore, of great importance to road engineers.

6.9. Research, both in the field and the laboratory, is called for in this subject which has not been given due attention in India. A countrywide survey of aggregates with a view to classifying them in respect of reactivity and sources of raw materials suitable for use as pozzolanas is necessary. The various research organisations in the country should take up immediately at least the work of assessing the pertinent qualities of aggregates available in areas under their jurisdictions. It is also necessary to explore extensively the sources of raw materials which can be economically employed for the manufacture of pozzolanas.

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Appendix 1

ALKALI REACTIVE MINERALS IN AGGREGATES

Sl. No.	Name	(a) Description & (b) Identifying characteristics	Mode of formation and occurrence
1.	Opal.	(a) Amorphous silica with variable amounts of water. Refractive Index (μ) and density varies with water. (b) μ between 1.40-1.46. Amorphous nature and low index important distinguishing characteristics. Not easy of identification petrographic examination of thin sections on account of its high transparency.	In igneous, sedimentary and metamorphic rocks. Always deposited by aqueous solutions except in case of granites. As veins and cavity fillings, in traps, mottles in granites, cementing matrix in sandstones and quartzites and coatings on gravels.
2.	Chalcedony.	(a) Fine grained, fibrous silica. (b) Conchoidal fracture, smooth waxy surfaces, concentric parallel fringes, μ less than 1.54.	Present in igneous and sedimentary rocks along with opal, cherts, flints, siliceous shales, slates etc. As cavity fillings, veins in traps and pebbles in gravel.
3.	Chert & Flint.	(a) Chert is fine grained, micro to cryptocrystalline quartz chalcedony or opal. Flint is chalcedonic. (b) Waxy dull appearance conchoidal fracture. Chert is whitish. Flint is dirty in colour.	In sedimentary rocks mostly limestone. Also in shales and as gravel.
4.	Tridymite & Cristobalite.	(a) High temperature modification of silica during the formation stage of certain igneous rocks. (b) Properties quite different from quartz. Tridymite has orthorhombic crystals μ - 1.469 and 1.473 along principal axes.	Igneous or metamorphic. Rhyolites, andesites, dacites, and related tuff. Can be present in slag aggregate.
5.	Acid & Intermediate Volcanic glasses.	(a) Alkali or Ferro-Alumino silicates with free silica in interstices. (b) Vitreous texture and chemical analysis for total silica content. SiO_2 less than 55 per cent - Basic glass. SiO_2 55 to 65 per cent - Intermediate glass. SiO_2 above 65 per cent - Acidic glass. μ - below 1.57 per cent for intermediate and acidic glass.	Rapid cooling of molten lava. Extreme forms are pitch stone, trass and pumice. Otherwise present uniformly in the mass of rocks such as rhyolites, dacites, andesites and related tuffs.

**“TIMING OF TRAFFIC SIGNALS FOR MIXED SLOW
AND FAST TRAFFIC”**

By

TEJBIR S. KHANNA*

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SYNOPSIS

Observations of behaviour of mixed traffic and the amount of time required for clearing through the crowded intersection of Queensway and Connaught Circus in New Delhi are presented. An analysis of the time required

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for different types of vehicles viz., mechanically propelled vehicles, cycles, tongas, bullock carts, to clear through the intersection is given.

Based on observations, the length of the cycle that can be used for time signals at the junction has been worked out. While stressing the need for further research, the Author concludes that special signals for right turning traffic are necessary when more than 50 per cent of the traffic is turning right or when the total time consumed by the straight going and right turning traffic from any one arm of the intersection on a green signal is more than the total time required for two separate signals for the straight going and right turning traffic. A list of items on which further research on the subject is needed is also included.

Calculations of time losses due to delay and suggested improvements for the intersection are included as Appendices.

1. INTRODUCTION

1.1. Traffic signals are electrically operated traffic control devices by which traffic is alternately stopped and allowed to go according to fixed (or variable) time schedule.

1.2. Traffic signals provide certainty of right of way, help movement of traffic in an orderly manner, thereby increasing the traffic carrying capacity of the street. Co-ordinated timing of various traffic signals in an area provides uninterrupted movement of traffic with no stops.

Fixed-time signal with single timing plan throughout the day may actually increase the average delay per vehicle if there is too much variation in traffic volume at different hours of the day.

Traffic signals should be installed and operated when abnormal delay to pedestrians or vehicular traffic occurs at a spot and when the traffic situation is beyond the capacity of a point duty traffic constable. Signals when properly operated do add to the efficiency of an intersection and thus safety is increased. Signals at intersection always reduce right-angle collisions but rear-end collisions, as observed in the U. S. A., are likely to increase. In general, the safety aspect alone, though very important, should not be the decisive factor for the installation and operation of such signals. Traffic signals should be installed mainly to regulate the flow of traffic and to add to the road and intersection efficiency, thus resulting in overall safer conditions.

1.3. Research on use of traffic signals for regulation of traffic has been done in western countries, but the results obtained there cannot be directly applied in India as both the road-users behaviour and the type of traffic here differ from those obtaining in the west. Specifications based on research, to serve as a yard-stick for assessing the need for a traffic signal and its timing under different traffic situations, are necessary. With this objective in view, research work on this subject has been taken up.

1.4. The method suggested by the Author and adopted for the study is very simple and applicable to existing conditions. But in no way it can be considered very accurate. This method should give only approximate results. The necessary equipment for accurate study is not available at present.

1.5. After a number of field observations, curves showing the green time consumption by different types of vehicles to enter and clear out of the intersection under study have been drawn. But these curves may be true only for the particular traffic conditions at the intersection. Only after studies at a fairly large number of intersections are complete, the relationship between similar curves can be found and green time consumption for different types of vehicles and different traffic conditions fixed.

2. PROCEDURE

2.1. The spot chosen for study is the crowded intersection of the Queensway and Connaught Circus near Scindia House, New Delhi, Plate I. At this junction, a traffic signal is already installed and manually operated by a police constable.

2.2. With a view to find out variation in the traffic intensity and the peak hours of traffic flow, half-hourly traffic counts were taken between 7-30 A. M. and 10-30 P. M. continuously for seven days, i.e., 29-4-55 to 5-5-55. During this period peak hours for traffic flow were found to be 9.00 to 11.00 A.M., 12-30 to 2-30 P. M. and 5.00 to 7.00 P.M. Therefore, further studies were carried out during these hours.

3. MIXED TRAFFIC

3.1. In India, for many years yet, it will not be possible to segregate slow and fast-moving traffic. So, the study has

been made under existing conditions and no efforts have been made to segregate traffic. At this intersection it has been observed that usually mechanically propelled traffic lines up in the right lane and the other traffic in the left lane, Photo 1.

3.2. The cyclists are mixed up in both lanes with majority of them in the left lane. Normally, time taken for different types of traffic to pass through the intersection would be less if they did not mix. But such mixing up cannot be avoided especially when a part of each type of traffic has to turn right and the rest has to go straight, Photo 2.

4. TIME-CYCLE-LENGTH

4.1. Research in western countries has proved that the shorter the cycle length the better the efficiency in traffic flow. But there is always a limit beyond which the length of the cycle cannot be reduced. Green time on each side is required to let at least the waiting vehicles pass through the intersection. When the green time interval on any side is not enough to allow all the accumulated waiting traffic to pass through the intersection, it is called a cycle failure.

4.2. To evolve a basis for fixing the cycle length of a traffic signal, it is necessary to find out the time required for different types of vehicles to pass through the intersection after starting from rest. In western countries, studies have been made on starting reaction time, successive reaction time, deceleration, acceleration, speed, delay, etc., but that is a long and cumbersome procedure though more accurate. In this study, observations, under existing traffic conditions, of time consumed by different types of traffic waiting for the green light to pass through the intersection have been made. For purposes of observation, vehicular traffic was divided into four categories, namely, (i) mechanically propelled vehicles, (ii) cycles, (iii) tongas, and (iv) bullock carts. Green time consumption, in seconds, for different vehicles standing in line waiting to cross the intersection was noted with a stop watch and plotted on graph sheets. Model values for each vehicle classified according to types mentioned above are given in Table I and plotted in Fig. 1.

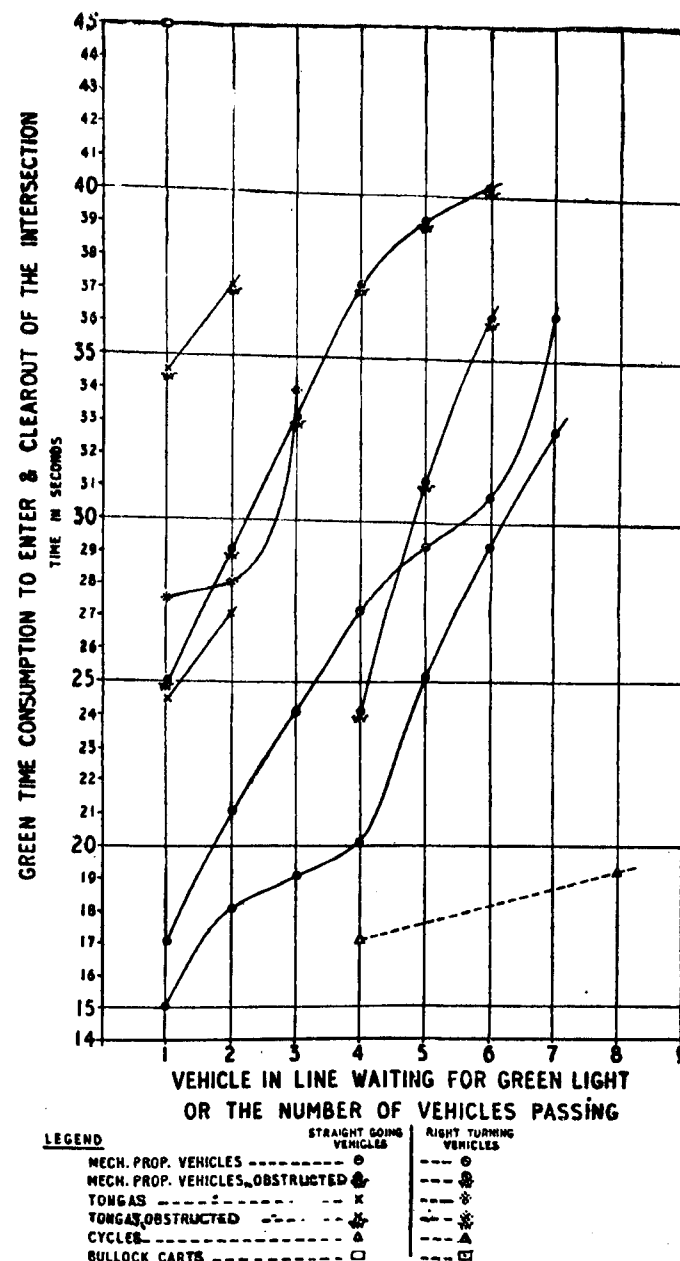


Fig. 1. Green time consumption to enter and clear out of the intersection for the various types of vehicles.

Vehicles first in line means first in its own category waiting to pass through the intersection.

TABLE 1
Green Time Consumption for Different Vehicles
Starting from Rest

Vehicle in line	Direction			
	Going straight	Going straight obstructed	Right turning	Right turning obstructed
Mechanically Propelled Vehicles				
1st	15.0	—	17.0	25.0
2nd	18.0	—	21.0	29.0
3rd	19.0	—	24.0	33.0
4th	20.0	24.0	27.0	37.0
5th	25.0	31.0	29.0	38.0
6th	29.0	36.0	30.5	40.0
7th	32.5	—	36.0	40.5
8th	—	—	—	—
Cycles				
	17.0	—	15.0	26.0
Tongas				
1st	24.5	—	27.5	34.5
2nd	27.0	—	28.0	37.0
3rd	—	—	33.8	—
Bullock Carts				
1st	45.0	—	40.0	50.0
2nd	—	—	40.0	—

5. ANALYSIS OF OBSERVATIONS OF MECHANICALLY PROPELLED VEHICLES

5.1. From Fig. 1 it can be seen that the curve for straight-going mechanically propelled vehicles changes slope after the 4th vehicle. This sudden change in slope shows that, under the conditions of traffic at the intersection, it is not advisable to have red time interval which may let more than four mechanically propelled vehicles accumulate.

5.2. Obstructed vehicles are those which had to appreciably slow down or stop in the intersection due to traffic obstruction before they clear through the intersection.

5.3. When there are more than four mechanically propelled vehicles waiting in line for the green light, the 5th, 6th, 7th, etc. vehicles are specially detained by the slow traffic turning right, Photo 3. The first four vehicles escape before slow traffic turning right starts causing hindrance.

5.4. The reason for extra additional time taken by the 5th, 6th, 7th mechanically propelled straight going vehicles is that they are slowed down by the mixing up of slow traffic which reaches the middle of the intersection in about 20 seconds (the time taken by the 4th vehicle to clear out) and part of it starts turning right. **For this reason time segregation of straight going and right turning traffic seems to be essential at this intersection.**

5.5. Further it can be seen from Fig. 1 that the time taken by the obstructed straight going vehicles to clear the junction is about 20 to 25 per cent more than that taken by the unobstructed vehicles. The main obstruction for straight going vehicles is the right turning traffic held up because of the opposite traffic, Photos 4 to 7.

5.6. The traffic turning right in turn hampers the flow of straight going traffic. First three straight going vehicles are not affected by this right turning traffic as there is space available on the left from where they can escape. But by the time the fourth vehicle reaches the centre, some of the slow moving traffic and cycles from the red side also reach the centre of the intersection leaving not much room on the left. **This is another reason why a special right turning signal should be provided to keep the straight going and right turning traffic segregated.** Further research work is necessary to fix limit on the amount of right turning and the straight going traffic, beyond which it would be necessary to have a special right

turning signal. In India, there seems to be a special need of such signals to avoid mixing up of slow and fast traffic at intersections.

5.7. Right turning mechanically propelled vehicles, as may be seen from Fig. 1, take more time to clear out of the intersection as compared to the straight going vehicles. This is because the vehicles have to slow down appreciably to take the right turn. Curve for the right turning vehicles rises steadily and an additional time of 3 to 4 seconds is consumed for each succeeding right turning vehicle. But the vehicles that are obstructed by the opposite traffic take about 45 to 50 per cent additional time to clear out of the intersection. These obstructed vehicles in turn hamper the flow of other traffic. The only way to reduce the delays would be to have a special right turning signal.

6. ANALYSIS OF OBSERVATIONS OF CYCLE TRAFFIC

6.1. When waiting for the green light, the cyclists stand right in front of the motor vehicles. They are at least aware of the danger involved in treading their way through the motor vehicles. They form a web through which it is almost impossible for the motorists to escape, Photos 8 and 9. Motorists can find their way out only after this web has cleared away. In fact, the cyclists behave in a most unpredictable manner. Since cycles do not stand in a line observations of cycle traffic could not be taken as intended.

6.2. Cyclists are a special cause of delay to motor vehicle traffic at intersections, Photos 10 to 12. If they formed themselves in platoons or groups of two each, standing one behind the other, they would create much less hazard for themselves and delay for the motor vehicles. That can be achieved only by education which takes a long time.

6.3. As may be seen from Table 1, it takes 17.0 seconds for a straight going cyclist starting from rest and only 15.0 seconds for the right turning unobstructed cyclist. This is because the cyclists do not have to slow down at the turn and, after they have taken the turn, the road is more clear because of the red light on this side. But the right turning cyclists which are obstructed by the opposite stream of traffic take 26.0 seconds to clear out. Thus about 65 per cent additional time is taken by the obstructed cycle turning right. Special right turning signal is also essential from cyclists' view point. Special research work is required on cycle traffic in India. In urban areas, cycle traffic

Photo 1.

Mechanically propelled traffic generally uses the right lane and other traffic the left lane.

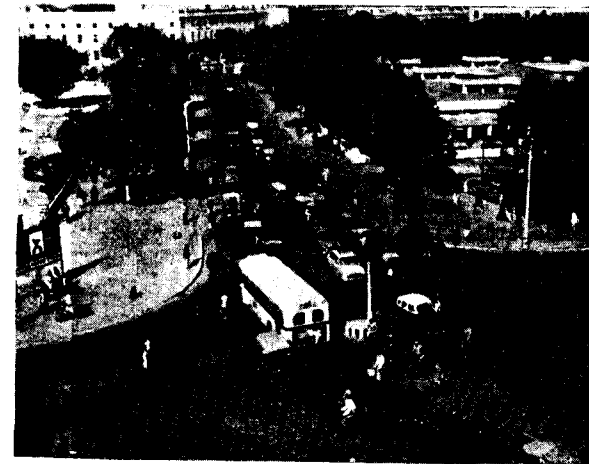
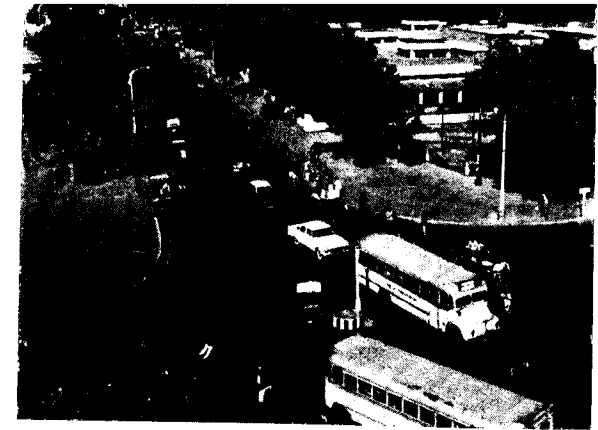
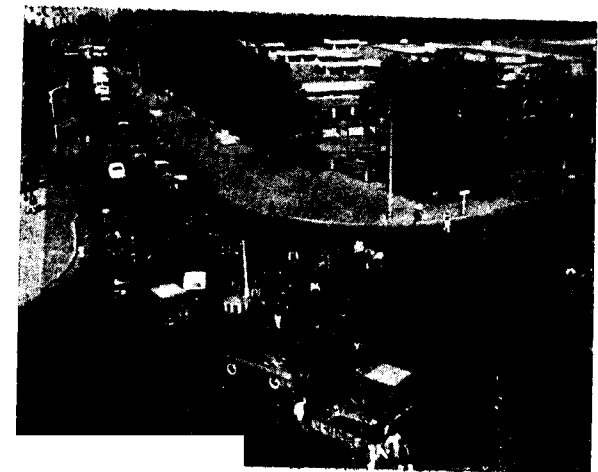


Photo 2.

There is a mix up of mechanically propelled traffic and slow traffic when slow traffic turns right.

Photo 3.

First 4 mechanically propelled vehicles escape before slow right turning traffic causes hindrance.



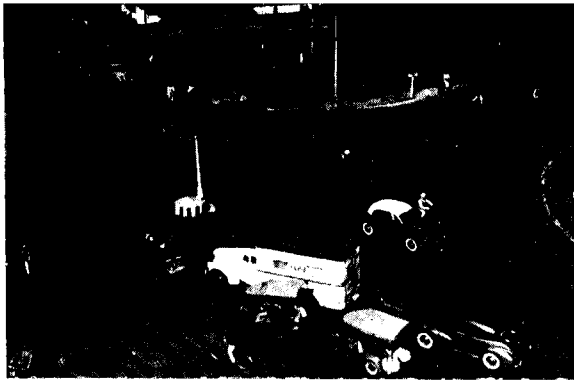
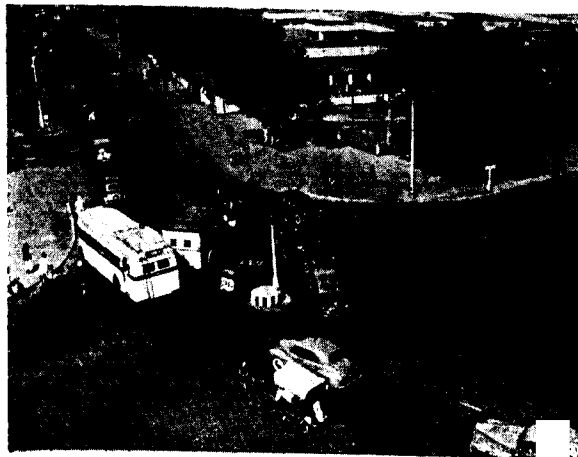


Photo 4.



Photo 5.



Photos 4, 5 & 6.

Right turning vehicles detained by the opposite traffic cause a great obstruction for the traffic behind them as may be seen from the Photos 4 to 7.

Photo 6.

Photo 7.
Right turning vehicles detained by the opposite traffic cause a great obstruction for the traffic behind them as may be seen from the Photos 4 to 7.



Photo 8.

Cyclists do not stand in a line while waiting for the green light.

Photo 9.
Cyclists form a web through which mechanically propelled vehicles cannot pass.



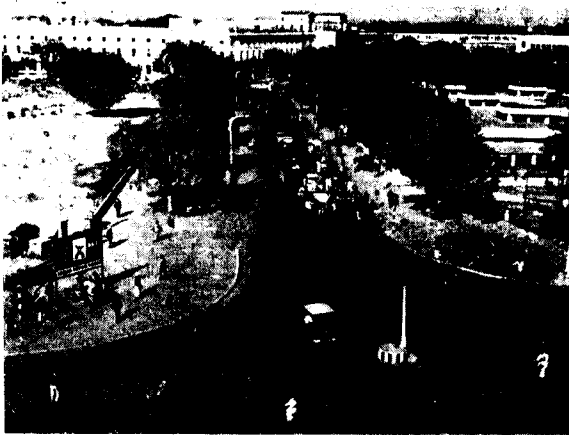


Photo 10.

Cyclists are a special cause of delay to motor vehicles and till they have cleared through the intersection motor vehicles are seriously obstructed by them. Photo 10 was taken on red light and Photo 11 on the following green light.

As may be seen the motor cycle-rickshaws escaped the cyclists but not the motor vehicles.



Photo 11.

Cyclists are a special cause of delay to motor vehicles and till they have cleared through the intersection motor vehicles are seriously obstructed by them.



Photo 12.

Cyclists are a cause of great obstruction to motor vehicles not only on the intersection but on the roads also.

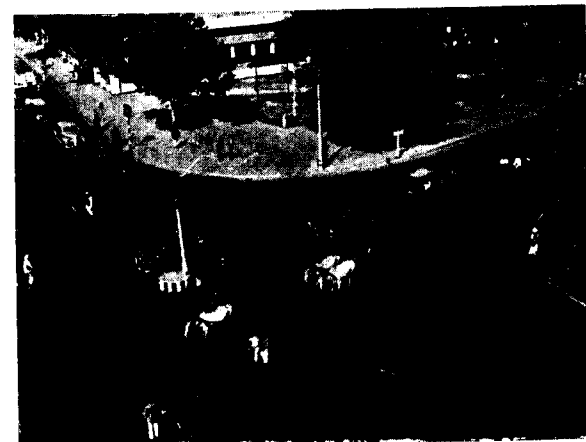


Photo 13.

Bullock carts take much longer than any other type of vehicle to clear through the intersection. During the peak hours bullock carts must be prohibited through signalled intersection.

Photo 14.

A caravan of bullock carts passing through the busy intersection is really a great obstruction for the faster traffic.

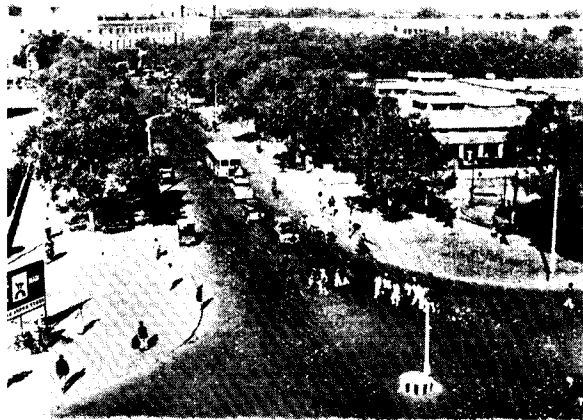
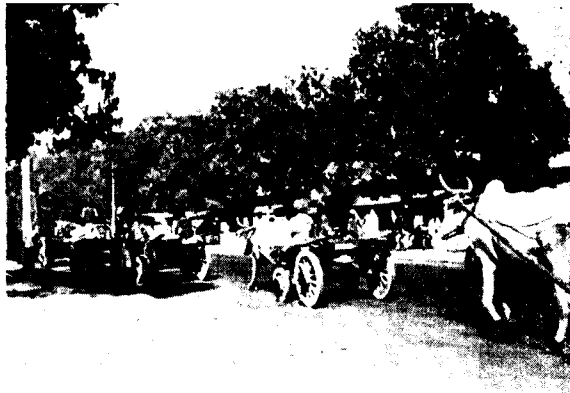


Photo 15.

Pedestrians do not use cross walks nor do they pay much attention to light signals.



Photo 16.

Animals make conditions even worse.

problem is as serious, if not more, as the bullock cart or tonga traffic problem.

6.4. At the present stage it is difficult to say as to what weightage should be given to the cycle traffic with respect to mechanically propelled traffic. For fixing the time-cycle-length it could be assumed that four cycles take about 17 seconds to clear out of the intersection and each additional four cycles take additional 2 seconds. Observations in the field show that the above mentioned timings hold good for right turning cycle traffic provided it is not obstructed. Observations to that effect were made in the field but not recorded as they were not very accurate.

7. ANALYSIS OF OBSERVATIONS OF TONGA TRAFFIC

7.1. Fig. 1 shows that the tonga first in line takes about 24.5 seconds to go straight through the intersection after starting from rest. The tonga second in line takes additional 2.5 seconds. Observations for 3rd, 4th, 5th, etc., tongas could not be made as there were not enough tongas accumulated during red light interval. They did not necessarily stand one behind the other. For all practical purposes, it would be safe to assume that each succeeding tonga would take additional 3 seconds.

7.2. It may be pointed out that one tonga is equivalent to about five mechanically propelled vehicles as far as the green time consumption is concerned and three tongas are equivalent to six mechanically propelled vehicles. However, this may not be true for calculating roadway capacity.

7.3. A right turning tonga takes 27.5 seconds against 24.5 seconds taken by the straight going tonga. Right turning tonga, second in line, takes an additional time of only 0.5 seconds. But for the third tonga the curve suddenly becomes steep showing that the third right turning tonga demands an additional time of 5.8 seconds over the second tonga. In the field it was observed that by the time the third tonga tried to make a right turn in the centre of the intersection, a thick stream of straight going traffic would come in its way. More observations are necessary to confirm this statement. Anyway, while timing the signals, effort should be made not to let more than two right turning tongas accumulate in one red light interval.

7.4. Right turning tongas obstructed by the opposite traffic take about 25 to 30 per cent additional time over that

taken by the unobstructed right turning tongas. This again shows that right turning signals would save this time.

8. ANALYSIS OF OBSERVATIONS OF BULLOCK CART TRAFFIC

8.1. It takes 45 seconds for a straight going bullock cart to clear through the intersection after the light has changed to green. This is equivalent to about ten straight going mechanically propelled vehicles and about seven straight going tongas. It has been observed that when once a longer cycle length is adopted to provide for one bullock cart, the vicious circle starts and all the following cycle lengths have to be longer resulting in unwanted delays, Photos 13 and 14.

8.2. A right turning bullock cart takes only 40 seconds against 45 seconds taken by the straight going bullock cart, the reason being the same as that for cycle traffic, see Para. 6.3. Right turning bullock cart obstructed by the opposite traffic takes about 25 per cent extra time.

9. ANALYSIS OF OBSERVATIONS OF PEDESTRIAN TRAFFIC

9.1. It has been observed that it takes about 11 seconds for a pedestrian to cross the road from one side to the other if he chooses to do so within the cross walk after the light has changed in his favour. Pedestrians did not form groups to cross the road at the correct spot and with the correct signal. Out of a total of 1,328 pedestrians observed on different week days, Fig. 2, during peak hours, 856 did not pay attention either to the signal or cross walk. In other words, about 64.5 per cent of the pedestrians walk carelessly causing traffic hazard, Photos 15 and 16. It was felt that wrong location of the pedestrian crossings is a contributory cause for pedestrians not using them. It has been observed that 39, 48, 53, 45 and 89 per cent of buses, passenger cars, motor cycle rickshaws, motor cycles and tongas respectively did not wait for their green light on the stop line. Unless all the vehicles are made to wait on the stop line, pedestrians cannot be expected to use the pedestrian crossing freely. When a high percentage of pedestrians are not using the crossing or paying any attention to the signal, it is difficult to allot a portion of the signal time for the pedestrians. But it would be quite safe to assume that 15 seconds are required for the waiting pedestrians to start and cross a 40 ft wide road unless there is an extraordinary rush. Further research work is necessary to fix the limit on the intensity of

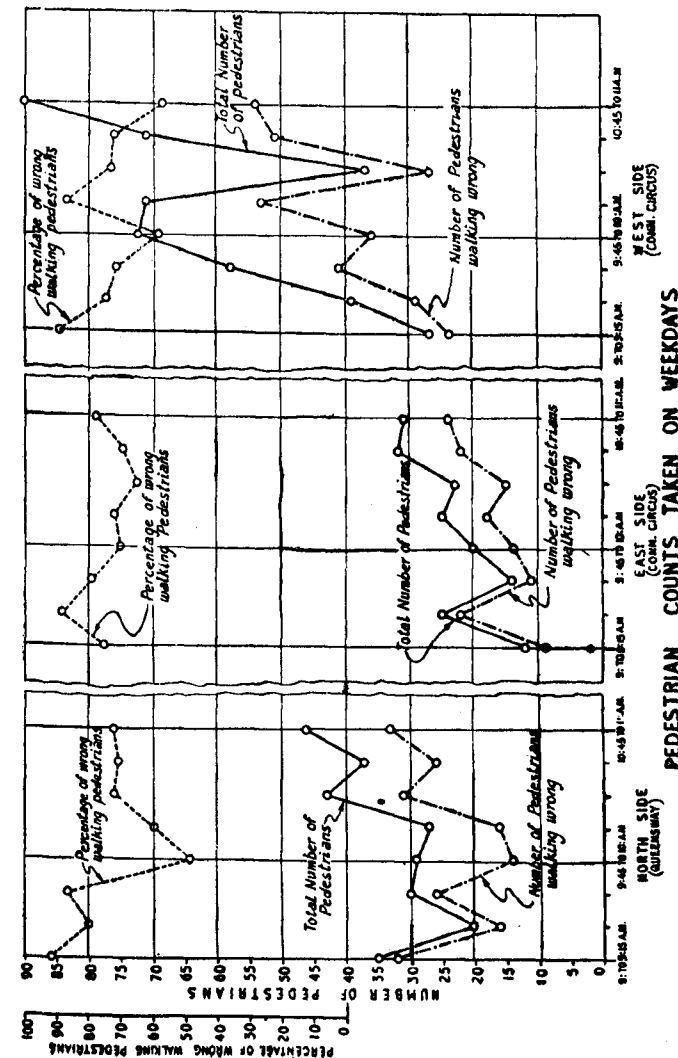


Fig. 2

pedestrian traffic beyond which special walk signal may be installed. Unless special walk signal is warranted, pedestrians should be allowed to cross the road with the green light in their favour. Under such conditions, green signals should not be less than 16 seconds specified by the American Specifications.

9.2. It has also been observed in the field that the vehicles turning left on red signal create hazard for the pedestrians walking on the pedestrian crossing with green light in their favour. For this reason left turns must be prohibited on red signal.

9.3. As far as green time consumption in entering and clearing out of the intersection is concerned after starting from rest, a comparison of different types of vehicles is given below.

1st tonga going straight unobstructed	= 5th mechanically propelled vehicle going unobstructed
	= 3rd mechanically propelled vehicle right turning unobstructed
	= 1st mechanically propelled vehicle right turning obstructed
2nd tonga going straight unobstructed	= 6th mechanically propelled vehicle going straight unobstructed
	= 4th or 5th mechanically propelled vehicle turning right unobstructed

This comparison may not be true for roadway capacity studies.

10. APPLICATION OF THE FINDINGS

10.1. Following is a suggested method for choosing the cycle length and distribution of green time for the two sides.

As mentioned in Para. 7.3, wherever tonga traffic is permitted minimum green time for that side will be = 27.5 - 5.0 (caution time) = 22.5 seconds.

If the shortest possible cycle, viz., of $22.5 + 22.5 = 45.0$ seconds + 10 seconds of caution time = 55.0, is chosen, the question of distribution of green time does not arise as a minimum green time of 22.5 seconds is necessary on each side. Distribution of green time out of the total cycle length would be necessary only when, at least, for one side 22.5 seconds of green time is not sufficient. In such cases, the minimum necessary green time would be given to that side and 22.5 seconds green to the other side. In case 22.5 seconds green time is not sufficient for either side, the green time should be distributed in the ratio of green time consumption for traffic for each side.

10.2. For judging the green time sufficiency under varying traffic intensity at different hours it would not be advisable to provide for the highest 15 minute period, because timing fixed on that basis would not be efficient during the whole day. So, for choosing the optimum cycle length 4 to 5 per cent cycle failure may be allowed. Cycle failure occurs when all the traffic accumulated during a particular red interval cannot be cleared during the following green interval. Supposing the signals are to be operated for 12 hours in which there are 48 intervals of 15 minutes each, allowing 4 per cent cycle failure two highest 15-minute periods may not be provided for.

Allowing 4 per cent failure, the amount of traffic for which the signals have to be provided for, as can be seen from the graph, Fig. 3, will be as follows :

	Connaught Circus		Queensway	
	Total	Percentage turning right	Total	Percentage turning right
Cycles	150	80	280	0
Mechanically propelled vehicles	90	50	60	10
Tongas	4	25	9	60

Checking the shortest cycle time, namely, 55 seconds, for the traffic conditions at the intersection, is given Section 11.

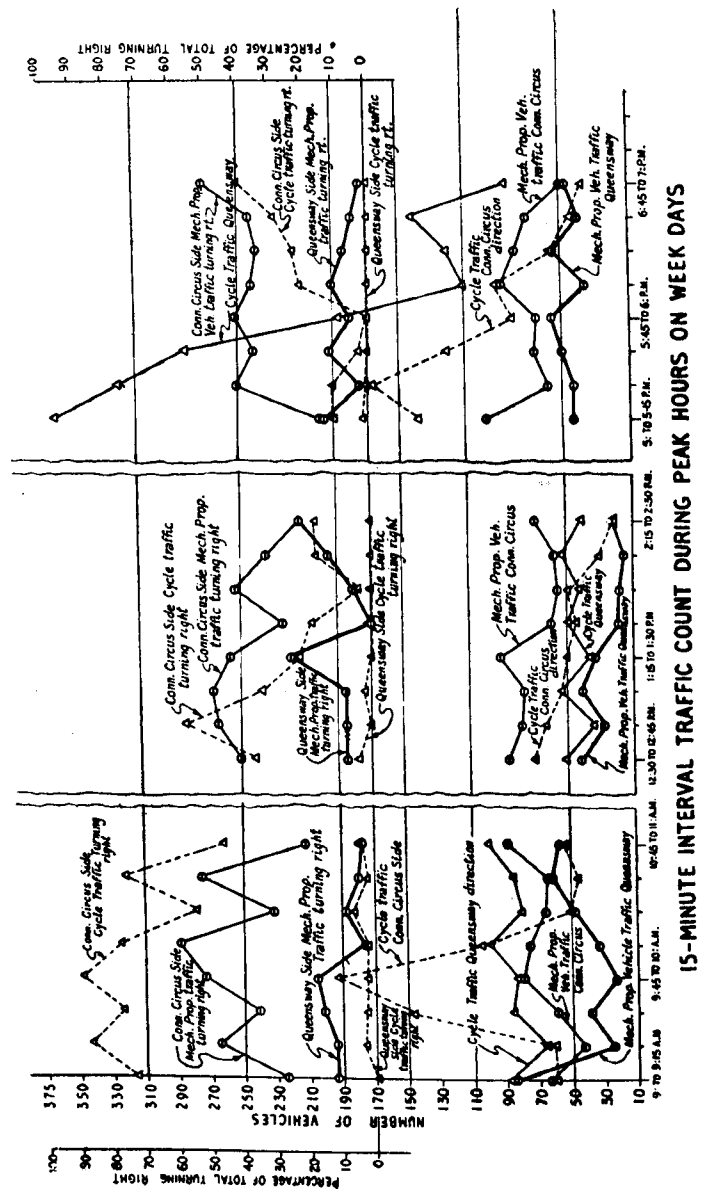


Fig. 3

11. CHECK FOR THE SHORTEST CYCLE, viz. 55 SECONDS

11.1. The total traffic expected from one side during the full cycle length will clear out during the green interval given on that side.

Traffic expected during the cycle length under trial, i.e., 55.0 seconds.

Total Vehicles	Connaught Circus side		Queensway side		
	Stopped vehicle	Total rightg turning	Total Vehicles	Stopped vehicle	Total right turning
Cycles					
$(150/900) \times 55$	6	8	$(280/900) \times 55 = 18$	11	0
Mechanically propelled vehicles					
$(90/900) \times 55 = 6$	4	2	$(60/900) \times 55 = 4$	3	1
Tongas					
$(4/900) \times 55 = 0.25$	1	1	$(9/900) \times 55 = 0.5$	1	1

(Out of 55.0 second cycle only 22.5 second is green time for each side. So, for 32.5 seconds vehicles are stopped. That gives the number of vehicles stopped during the cycle.)

Readings for the required time can be taken straight away from Fig. 1.

11.2. Connaught Circus Side Cycle traffic check

Ten cycles can pass in 21.0 seconds. This is less than 27.5 seconds. Therefore the timing is suitable

11.3. Mechanically Propelled Vehicles

Four vehicles starting from rest will take 20.0 seconds to clear out. It has been observed in the field that the average minimum headway in the free flowing mechanically propelled vehicles is 4.0 seconds. With this minimum headway, additional 4 seconds are required for each free flowing vehicle. Therefore, 4 waiting and 2 free flowing mechanically propelled vehicles should take 28.0 seconds to clear out. 28 seconds are very nearly equal to 27.5 seconds, Fig. 4.

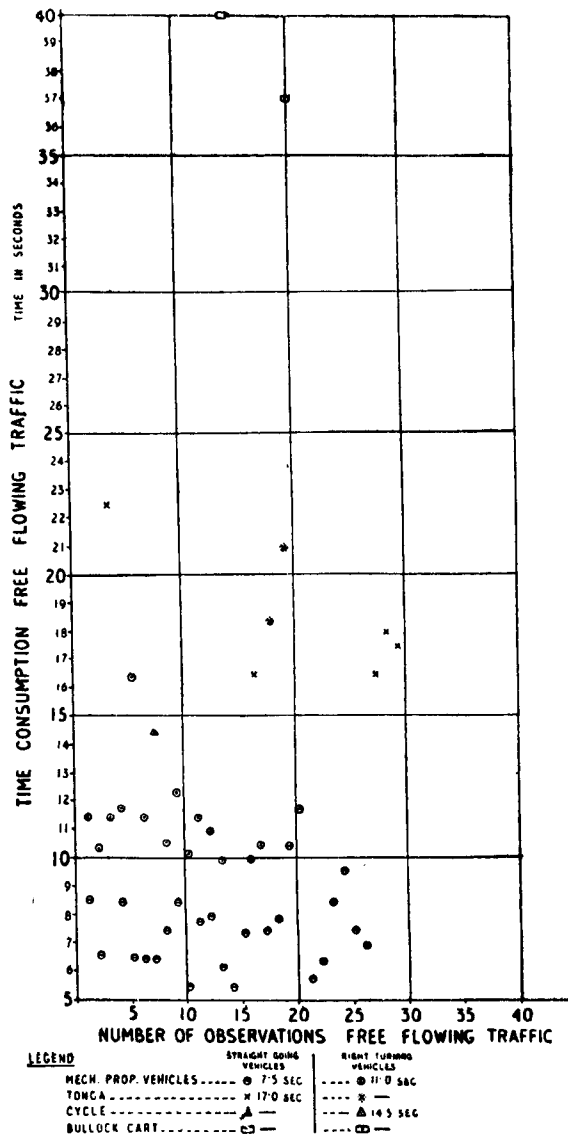


Fig. 4

11.4. Entrance Time Check

Now total time taken by all these 6 mechanically propelled vehicles should be less than the green time interval, i.e., 22.5 seconds. Time taken for 4 mechanically propelled vehicles waiting in line to enter the intersection, as may be read from Fig. 5, (modal values are plotted on graph), is 12.5 seconds. Allowing 4 seconds for each additional free flowing vehicles,

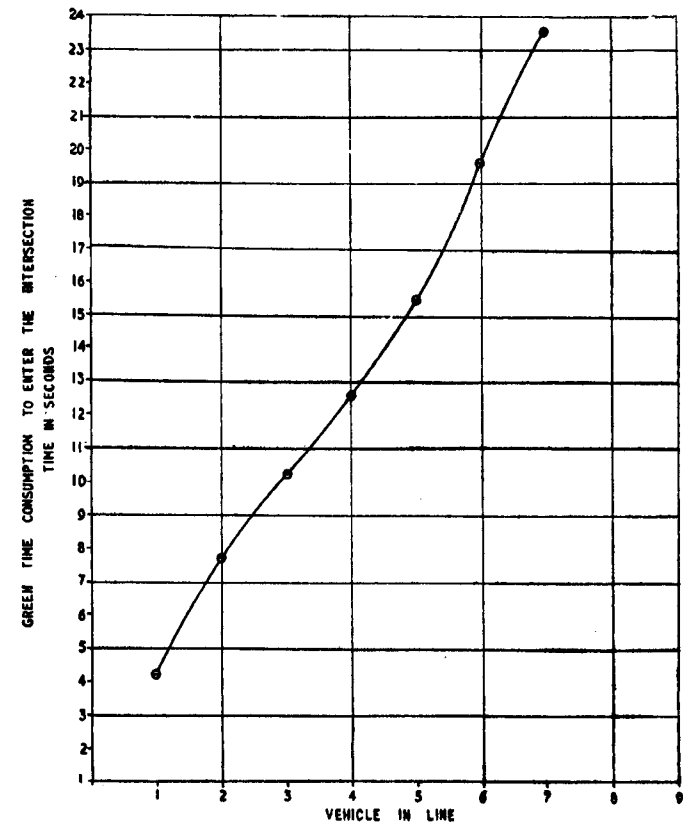


Fig. 5. Green time consumption to enter the intersection for different vehicles.

the total time taken for all the 6 vehicles would be $12.5 + 8.0 = 20.5$ seconds. 20.5 is less than 22.5.

11.5. Right Turning Check

Suppose out of the waiting vehicles, the 4th vehicle is turning right. 27.0 seconds will be required for this vehicle to clear out of the intersection, Fig. 1. For a free flowing right turning mechanically propelled vehicle it takes 11.0 seconds, Fig. 4, which means that about 6.0 seconds before clearing out the right turning vehicles had already taken a right turn. After taking the turn, the vehicle does not consume the green time or caution time of that side. 27.0 minus 6.0 = 21.0 are less than 27.5. Now, further, suppose that second free flowing vehicle is going to turn right. Time consumed by this vehicle to enter the intersection will be $12.5 + 8.0 = 20.5$. Since it takes 11.0 seconds for the free flowing vehicle to clear out of the intersection, it will take approximately 6.0 seconds to reach the middle and take a right turn. Total time taken before it turns right will be $20.5 + 6.0 = 26.5$ seconds. 26.5 is less than 27.5.

11.6 Tonga Check

One tonga turning right waiting for green light will take 27.5 seconds.

11.7. Check on Queensway Side

18 cycles will take 25.0 seconds to clear out. 25.0 is less than 27.5.

All other traffic on this side is less than the traffic on Connaught Circus side; therefore, 22.5 second green time with 5 seconds caution should be alright.

So, at this particular intersection 55.0 second cycle should work well. Out of the 55 seconds, 22.5 second green time and 5.0 second caution time may be given to each side.

11.8. Pedestrian Check

It may be seen from Fig. 2 that over 60 per cent of pedestrians either do not pay any attention to signal or to the existence of the pedestrian crossing. Under such conditions it is very difficult to allot any pedestrian crossing time or making a check on this time when it runs concurrently with other traffic 'Go' period.

Further, it has been observed in the field that it takes 10 to 11 seconds for a pedestrian to cross the roadway from one end to another in a pedestrian crossing. Width of the roadway is 40 ft

which means that pedestrian walks at a speed of about 4 ft per second.

According to the American Standards, green time should be at least 16 seconds for 40 ft width where there is no special pedestrian rush. 22.5 seconds is greater than 16.0 seconds.

11.9. Equitable Distribution of Green Time

Supposing 55 seconds cycle length is not enough, then 60 seconds and 65 seconds etc. cycle length may be tried.

Green time division may not necessarily be equal for both sides. Green time distribution should be made according to the traffic needs from each side.

Assuming a cycle length of 70 seconds, with 5 second yellow interval for each side, 60 seconds of total green time has to be distributed. Suppose 30 seconds of green time is given to each side.

In 70 seconds, on Queensway side the expected traffic =
Cycles $(280/900) \times 40 = 13$ stopped + 9 flowing.
Turning right 9.

Mechanically $(60/900) \times 40 = 3$ stopped + 2 flowing.
propelled vehicles Turning right 1 (controlling factor)

Tongas $(9/900) \times 40 = 0.4$ stopped + 0.1 flowing.
Turning right 1.

Connaught Circus Side

Cycles $(150/900) \times 40 = 7$ stopped + 5 flowing.
Turning right 10.

Mechanically $(90/900) \times 40 = 4$ stopped + 3 flowing.
propelled vehicles Turning right 4 (controlling factor)

Tongas $(4/900) \times 40 = 0.2$ stopped + 0.2 flowing.

From Queensway side out of 5 mechanically propelled vehicles, one is turning right.

Time consumed for 4 straight going vehicles is 20.0 seconds and one right turning unobstructed is 17.0 seconds.

From Connaught Circus side, out of 7 mechanically propelled vehicles 4 are right turning. For 3 straight going vehicle time

consumed is 19.0 seconds and for four right turning vehicles time consumed is 27.0.

So green time for the two sides may be divided in the ratio 20 : 27 (as Queensway : Connaught Circus) and check may be made.

12. SPECIAL RIGHT TURNING SIGNAL

12.1. The calculations in Section 11 have been made on the assumption that no right turning vehicle will be very much obstructed by the opposite stream of traffic. But this assumption can be correct only up to a certain percentage of traffic turning right beyond which special provision must be made for the right turning traffic. It is difficult to specify the maximum percentage beyond which a special right turning signal should be provided but, as observed in the field, appreciable delay is caused to the vehicles turning right from Connaught Circus side to the Queensway side and it is felt that a right turning signal is needed on this side.

12.2. As may be seen from Fig. 3 the percentage of vehicles turning right from Connaught Circus side to Queensway side is fairly high. The percentage of mechanically propelled vehicles turning right varies between 25 and 60 and that for cycles varies between 90 and 50 in the morning and between 0 and 80 per cent in the evening. It has been observed in the field that no appreciable delay is caused to traffic turning right from Queensway side. The percentage of mechanically propelled vehicles turning right from this side varies between 0 and 20 and that of cycles turning right varies between 0 and 5.

12.3. From the study it is seen that a special right turning signal is necessary when the percentage of mechanically propelled right turning vehicles is between 25 and 60 (on the average, about 40 per cent or above as may be seen from Fig. 3). For cycles traffic the percentage turning right should vary between 50 and 90. For traffic turning right to the Connaught Circus side, i.e., mechanically propelled vehicles 0 to 20 per cent and cycles 0 to 5 per cent, it can be safely suggested that no right turning signal is required. So the limit beyond which the right turning signal is required is certainly above 20 to 25 per cent traffic turning right but it is difficult to draw a line of demarcation even though approximate after the field study of only one intersection.

12.4. Installation of a 'special right turning signal' may be justified only when one or a combination of the following conditions exist:

- (1) When the total time consumed by the straight going and right turning traffic on a green signal is more than the total time required for the two separate signals, i.e., the signal for straight going traffic and the signal for right turning traffic. Only after a large number of field observations the maximum percentage of traffic turning right beyond which a special right turning signal should be installed be fixed.
- (2) When a special right turning signal is installed, at least one lane has to be reserved for the right turning traffic. So, on a two-lane roadway special right turning signal cannot be given. Installation of the right turning signal is possible only on a four lane roadway where two lanes are carrying traffic in each direction. The installation of a special right turning signal is possible, only if one lane out of these two lanes could be reserved for the right turning traffic. Logically speaking, to reserve one out of the two lanes for right turning traffic there should be about 50 per cent or more of the traffic turning right.
- (3) When mixed slow and fast traffic uses a road, it is observed that the left lane is generally occupied by slow moving traffic and the right lane by the fast traffic. If the right lane is reserved for the right turning traffic then all straight going fast vehicles would not like to stand behind the slow traffic. But at an intersection where only fast and cycle traffic is permitted, there is a possibility of installation of special right turning signal even if there are only two lanes for traffic in one direction. But when slow and fast traffic is permitted to pass through the intersection with only two lanes in each direction and a special right turning signal is justified from the time consumption point of view, an additional lane about 100 to 150 ft long may be provided beyond the intersection on the arm for which a special right turning signal is required.

13. CONCLUSIONS

13.1. Installation of traffic signals is necessary during the peak hours of traffic flow through this particular intersection.

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13.2. Further research work is required to formulate standards for the installation of signals.

13.3. Special right turning signal may be installed under any of the following two conditions :

- (i) when more than 50 per cent traffic is turning right, and
- (ii) when total time consumed by the straight going and right turning traffic, from any one arm of a particular intersection, on a green signal, is more than the total time required for two separate signals for straight going and right turning traffic separately.

13.4. Where special right turning signal is installed it is desirable to have a 100 to 150 ft long storage lane on the intersection arm.

13.5. Bullock carts must be prohibited through an intersection where the traffic is controlled by light signals. Bullock carts may be permitted to pass through the intersection during night and when light signals are not in use.

13.6. Separate cycle tracks are necessary on urban roads. Such tracks should be shown on intersection also by road-markings.

13.7. Control of movement of pedestrians is very desirable on the intersection. Since a high percentage, i.e., about 65 per cent pedestrians ignore the pedestrian crossing or the light signal, special measures should be taken to control them. If even after the relocation of pedestrian crossings, pedestrians continue walking through the intersection or outside the pedestrian crossing, railings along the kerb at the intersection are recommended so that pedestrians may not be able to enter the intersection in any manner they like. The railing should be open at the crossing only.

13.8. On red light no vehicle should be allowed to cross the stop line to enable the pedestrians to use the pedestrian crossings.

13.9. Left turns on red light should be prohibited to encourage the pedestrians to walk safely with green light in their favour. Not permitting left turns through red light would also eliminate the disturbance caused to the slow moving traffic waiting in the left lane by the faster vehicles waiting to take left turn through red light.

13.10. When the signals are manually controlled, effort should be made, when four mechanically propelled vehicles

have accumulated, to switch the lights to green. The smaller signal time-cycle lengths will result in less delay.

13.11. A minimum of 5.0 seconds caution time is recommended for the slow moving traffic to clear out of the danger of sideways collision. Slow, free flowing traffic, i.e., tonga or cycle, take about 17 seconds to clear out of the intersection. Supposing one second after a tonga has entered the intersection the green light changes to caution for 5 seconds after which the traffic from the other direction is released, the tonga will have travelled for 6 seconds before the traffic from the other side is released. Minimum time taken for a mechanically propelled vehicle first in line to reach the middle of the intersection will be about 7.0 seconds (since it takes 15.0 seconds for such vehicles to clear out). So, by the time green light traffic reaches the middle of the intersection, the tonga will have travelled for 13 seconds whereas it takes only 17.0 seconds to clear out of the intersection, i.e., the tonga will have travelled $\frac{2}{3}$ rd of the intersection and thus got out of the danger of sideways collision.

13.12. Where tonga traffic is permitted, minimum cycle length must not be less than 55.0 seconds which includes 22.5 seconds green time for a right turning tonga from either direction and a 5.0 second caution interval after the green light $(22.5 + 5.0) \times 2 = 55.0$ seconds.

13.13. Tonga traffic may also be prohibited during the peak hours unless the intensity of tonga traffic is less than 15 numbers in a 15-minute interval.

13.14. Further Research Studies

(1) Study, in consultation with the traffic police, is required to fix a limit on the amount of traffic beyond which traffic signal should be installed.

(2) Research is necessary to fix a limit on the amount of both right turning and straight going traffic beyond which it would be necessary to have a special right turning signal.

(3) A detailed study of cycle traffic especially in urban areas is very essential.

(4) Study of pedestrians at intersections is essential to find out the intensity of pedestrian traffic that may warrant a special pedestrian signal, under different traffic conditions.

(5) Research is necessary to fix a limit on the amount of tonga traffic below which this type of traffic should be prohibited from passing through the signalised intersections during the peak hours.

Study on 'timing of traffic signals' at other crossings with varying traffic intensities and percentages of different types of traffic would yield data for timing the length of the cycle for different conditions.

13.15. While making observation at the crossing it was noticed that the crossing needed some physical improvements from traffic engineering point of view. The suggested improvements are given in Appendix 1, and shown in Plate I. The improvements suggested are in conformity with the needs of mixed traffic at the particular crossing. After these improvements are made, timing study will again be carried out at this crossing.

13.16. Calculations of delay losses to the mechanically propelled vehicles are given in Appendix 2. Field observations were taken to calculate delay losses. Delay loss per vehicle has been calculated for the proposed signal cycle length of 55.0 seconds as against the observed cycle length of 90 seconds to 110 seconds during peak hours. As is evident from the calculations, delay loss per vehicle for 90 seconds or 110 seconds cycle should be far higher.

ACKNOWLEDGMENT

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Appendix 1

SUGGESTED IMPROVEMENTS *

The following changes are suggested in order to improve the traffic safety and efficiency on this intersection, Plate I.

1. Control island should be removed. At present right turning vehicles have to go round this island which makes the right turning path long and tedious. Without this island right turning path would be short and much less time would be consumed by the right turning vehicles, thereby, adding to the efficiency of the intersection.

2. Unwanted openings on the curbs must be closed. There are two gaps in the curb on Scindia House side. That gives this intersection two additional approaches which reduce efficiency of the intersection. These gaps as well as other places, where the curbs have been bevelled with the road for cars to be driven up, affect safety of the pedestrians using the side-walks.

3. Lanes, cross-walks, stop-lines should be re-marked as shown in Plate I. Special return markings, as shown would be desirable after removal of the central island.

Cross-walk should be located in line with the side-walk as much as possible otherwise pedestrians do not like to use them. Proposed location is shown in Plate I under Suggested Improvements.

4. Light signals are not properly located as it is difficult for the pedestrians to see the green light favouring them in the cross-walks. Light signals will have to be shifted to suit the new location of cross-walks.

5. As result of the study it has been found desirable to have a special right turning signal on Connaught Circus side. Much traffic turns right from Connaught Circus to Queensway. So a storage lane about 100 to 150 ft long, may be constructed on each arm on Connaught Circus. All the right turning traffic may be required to stop in the extreme right lane and to move only on the right turning signal.

6. It has been observed that over 60 per cent of the pedestrians do not pay any attention to the signals or cross-walks and enter the intersection from any spot which is dangerous for the pedestrians themselves and also reduces the efficiency of the intersection.

It is suggested that railings may be fixed along the curb on the intersection. The railings should be open at the cross-walks only so that pedestrians cannot enter the intersection and are obliged to walk on the cross-walks only.

* It may be pointed out that no detailed study was made with a view to improve this particular intersection.

Appendix 2

CALCULATION OF DELAY LOSSES*

Potential Blocking Period. This is the time during which a vehicle is potentially blocked by red signal or by the vehicles ahead which have been delayed by the signal.

The pattern of delay losses is shown in Fig. 6.

Mechanically Propelled Vehicles: For the first vehicle which arrives at the intersection, the potential blocking period is equal to the red interval 'r' of the signal though there is a chance that the vehicle may not have the full potential if it arrived after the beginning of the red interval. Even if never stopped, the second vehicle on entering the intersection, may not follow closer than 1.0 seconds (1.0 is the minimum spacing observed between 2nd, 3rd and 4th vehicles, Fig. 4. In the U.S.A. the spacing value taken is 1.7 seconds which is the spacing at a speed of 15 miles per hour. This value of 1.0 should be checked again before it is finally accepted).

The potential blocking period for the second vehicle is

$$r + 4.2 + 1.0 = r + 5.2$$

Potential blocking period for the 3rd vehicle is

$$r + 4.2 + 3.5 + 1.0 = r + 8.7$$

Potential blocking period for the 4th vehicle is

$$r + 4.2 + 3.5 + 2.5 + 1.0 = r + 11.2$$

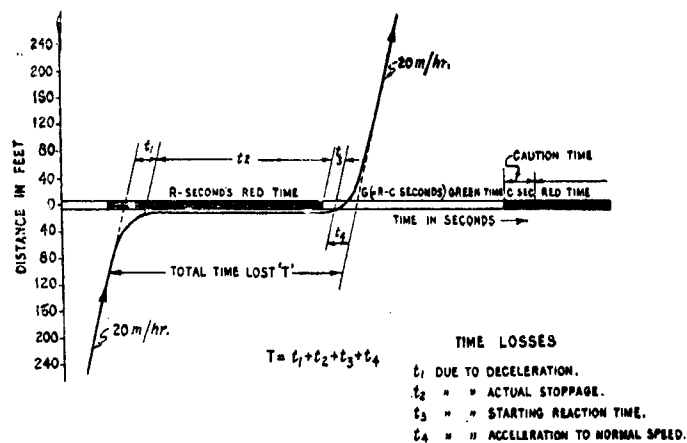
Potential blocking period for the 5th vehicle is

$$r + 4.2 + 3.5 + 2.5 + 2.3 + 1.0 = r + 13.5$$

In general the potential blocking period is obtained by adding to the red signal interval the additional delay intervals caused by preceding vehicles plus 1.0 second.

Fig. 7 shows the additional blocking period that is created when various numbers of vehicles are retarded.

* The method used for calculating delay losses is as suggested in Technical report No. 1, Yale Bureau of Highway Traffic.



TIME LOSS STUDY OF THE FIRST VEHICLE STOPPED BY RED LIGHT

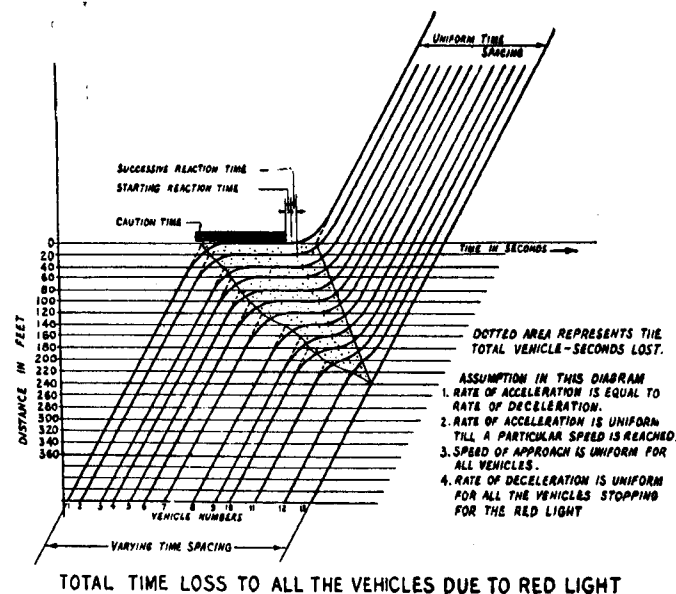


Fig. 6

Green Time consumed to Enter the Intersection starting from rest

Vehicle No.	Mechanically propelled vehicles		Tongas		Bulluck carts	
	Time interval between vehicle	Entrance time after light changes to green	Time interval between vehicle.	Entrance time after light changes to green	Time interval between vehicle	Entrance time after light changes to green
1.	—	4.2	—	6.5	—	9.0
2.	3.5	7.7	3.5	10.0	3.5	12.5
3.	2.5	10.2	—	—	—	—
4.	2.3	12.5	—	—	—	—
5.	2.9	15.4	—	—	—	—
6.	4.1	19.5	—	—	—	—
7.	4.3	23.5	—	—	—	—
8.	—	—	—	—	—	—

Practical Method of Determining Number of Vehicles Retarded at the Intersections

It may be remembered that the percentage of vehicles retarded is not the same as the percentage of red signal to the total cycle. The proportion of the vehicles retarded is greater than the proportion of the red signal to the entire cycle. Since each retarded vehicle in turn adds to the blocking period.

Fig. 7 gives the additional blocking period when different number of vehicles are retarded.

Connaught circus side

$$\text{Mechanically propelled vehicles} \left\{ \frac{90}{900} = 0.10 \right.$$

Queensway side

$$\frac{60}{90} = 0.67$$

Stop time or red signal on Connaught Circus side is
 $22.5 + 10.0 = 32.5$ seconds.

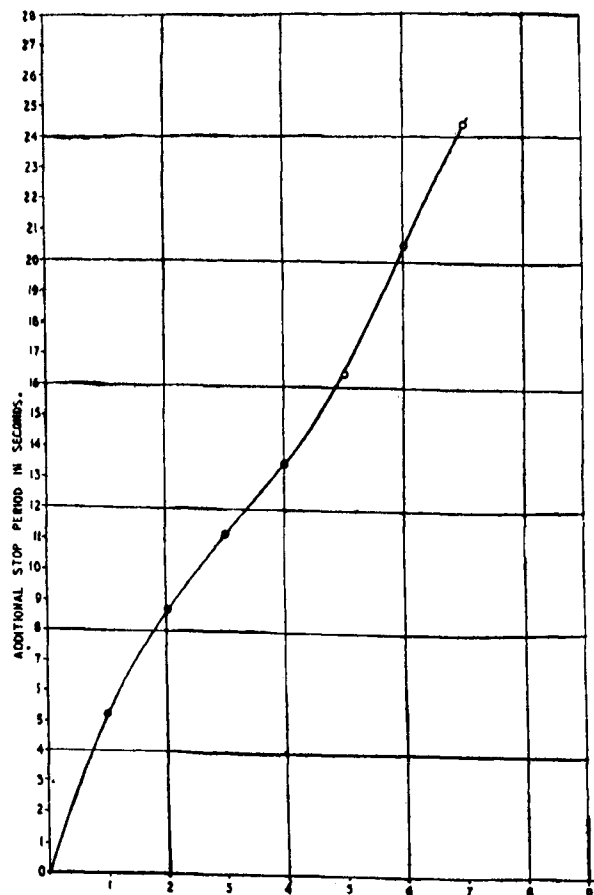


Fig. 7. Additional blocking period created when various numbers of vehicles are retarded

Therefore mechanically propelled vehicles retarded equal to $32.5 \times 0.1 = 3.25$ which means 3 for each of three cycles and 4 for the fourth cycle.

Now these 3.25 vehicles tend to increase the effective length of red signal. Referring Fig. 7 it can be seen that 3.25 vehicles increase the blocking period by 11.8 seconds. The total blocking period may now be considered to be $32.5 + 11.8 = 44.3$ seconds. This blocking period will retard $44.3 \times 0.1 = 4.43$ vehicles.

Again the increase in the blocking period due to 4.43 vehicles will be 14.7, Fig. 7, and the total blocking period may now be taken as $32.5 + 14.7$ seconds (This is third approximation).

It has yet to be decided whether full caution signal after the green light should be included with red signal or not. Field observations are required to find out the percentage of caution time to be included with red time.

Average number of vehicles stopped with 90 mechanically propelled Vehicles 15 per minute per lane and 32.5 seconds red period.

	Length of blocking period.	Average number of vehicles retarded
1st Approx.	32.5	3.25
2nd „	44.3	4.43
3rd „	47.2	4.72
4th „	47.9	4.79
5th „	48.2	4.82
6th „	48.2	4.82

It would be sufficiently accurate to use an average of 4.82 vehicles per red signal which corresponds to 48.2 seconds of blocking period even though the red signal is only 32.5 seconds.

Average Arrival Methods (Fig. 8)

The red interval is 32.5 seconds. It may be assumed that the first vehicle will arrive on the average at the mid-point, wait 16.25 seconds and adding the entrance time requirements it will lose 4.2 seconds in entering the intersection.

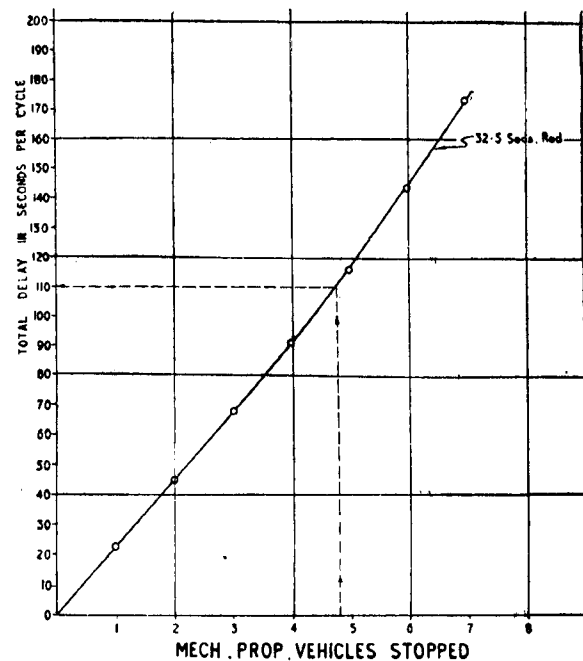


Fig. 8. Delay for various numbers of vehicles retarded—Average arrival method.

Another 2.0 seconds may be added for acceleration.

The total loss is $= 16.25 + 4.2 + 2.0 = 22.45$ seconds

The second vehicle arrives on the average mid-point of the stop period of $r + 5.2$ and leaves at $r + 7.7$. The time loss is therefore

$$[r + 7.7 - (r + 5.2) / 2] + 1.0 = 32.5 + 7.7 - \frac{(32.5 + 5.2)}{2} + 1.0$$

$$= (40.2 - 18.9) + 1.0 = 22.3 \text{ seconds.}$$

1.0 second is added as acceleration loss.

The loss for the third vehicle is

$$r + 10.2 - (r + 8.7) / 2 + 1.0$$

$$= 32.5 + 10.2 - (32.5 + 8.7) / 2 + 1.0$$

$$= 42.7 - 20.6 + 1.0 = 23.1 \text{ seconds.}$$

Total loss of time for 3rd vehicle per cycle is

$$= 22.45 + 22.3 + 23.1 = 67.85 \text{ seconds.}$$

$$\text{The loss for the fourth vehicle} = r + 12.5 - (r + 11.2) / 2 + 0$$

$$= 45.0 - 21.9 = 23.1 \text{ seconds.}$$

Acceleration loss is not added for the 4th, 5th etc. vehicles as it is presumed that they have already accelerated before entering the intersection.

$$\text{The loss for the fifth vehicle} = r + 15.4 - (r + 13.5) / 2 + 0$$

$$= 47.9 - 23.0 = 24.9 \text{ seconds.}$$

$$\text{The loss for the sixth vehicle} = 32.5 + 19.5 - (32.5 + 16.4) / 2$$

$$= 52.0 - 24.5 = 27.5 \text{ seconds.}$$

$$\text{The loss for the 7th vehicle} = 32.5 + 23.5 - (32.5 + 20.5) / 2$$

$$= 56.0 - 26.5 = 29.5 \text{ seconds.}$$

$$\text{Total loss of time for 7 vehicle per cycle} = 67.85 + 23.1 + 24.9 + 27.5 + 29.5 = 172.85 \text{ seconds.}$$

From Fig. 8 total delay for 4.8 vehicles is 110.0 seconds per cycle when red signal is 32.5 seconds.

From the total delay of 110.0 seconds for 4.8 mechanically propelled vehicles delay per vehicle for a red signal of 32.5 seconds per cycle length of 55.0 seconds can be calculated.

There will be $(3600/55 = 65.5)$ 65.5 red intervals per hour. So total delay per hour will be $65.5 \times 110 = 7200$ seconds.

There are 90 Mechanically Propelled Vehicles per 15 min. or 360 per hour.

$$\therefore \text{Average delay per vehicle} = 7200 / 360 = 20 \text{ seconds.}$$

In general, the delay per vehicle for any signal setting may be found by a formula

$$D_v = \frac{D_r \cdot N}{V}$$

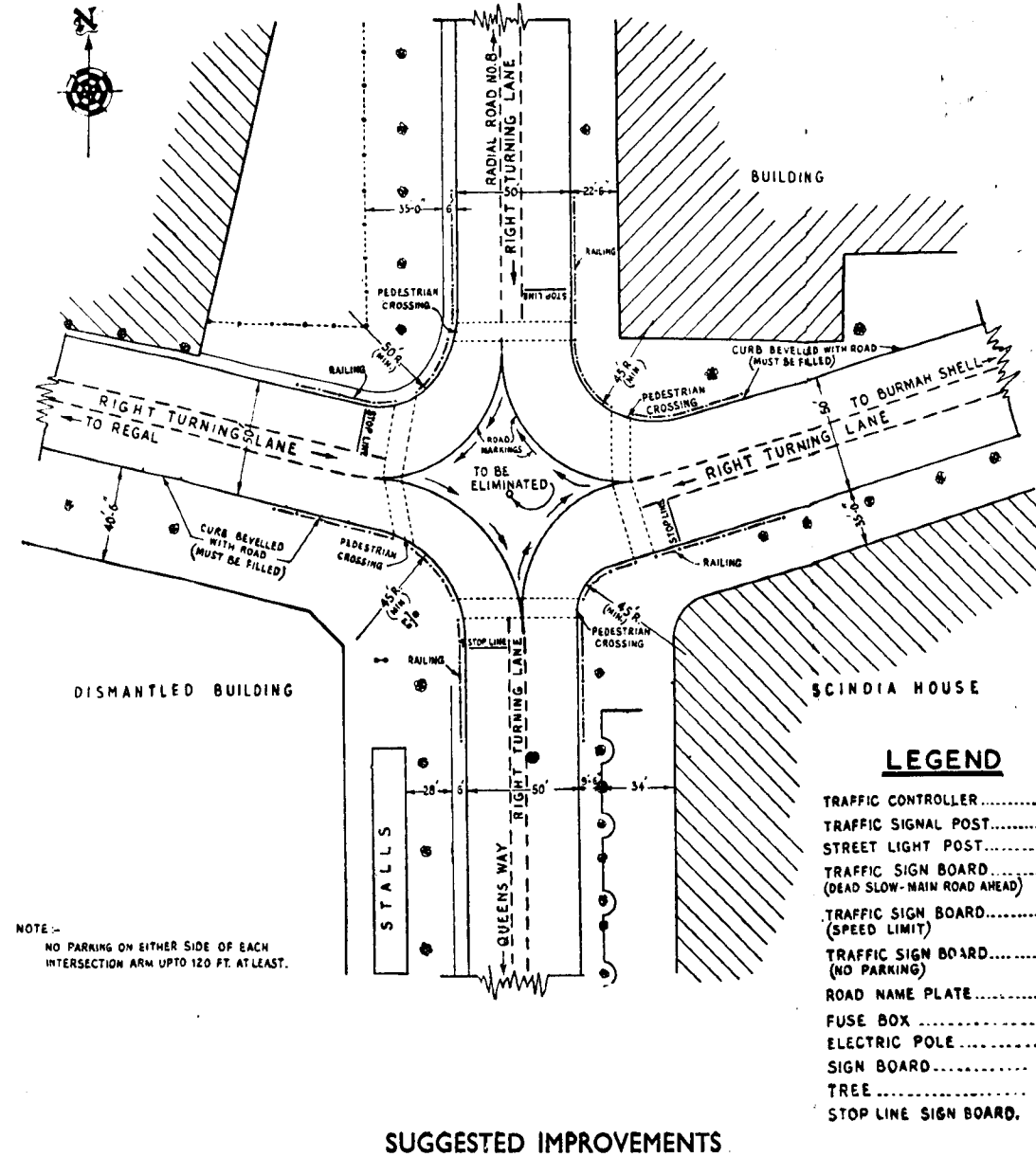
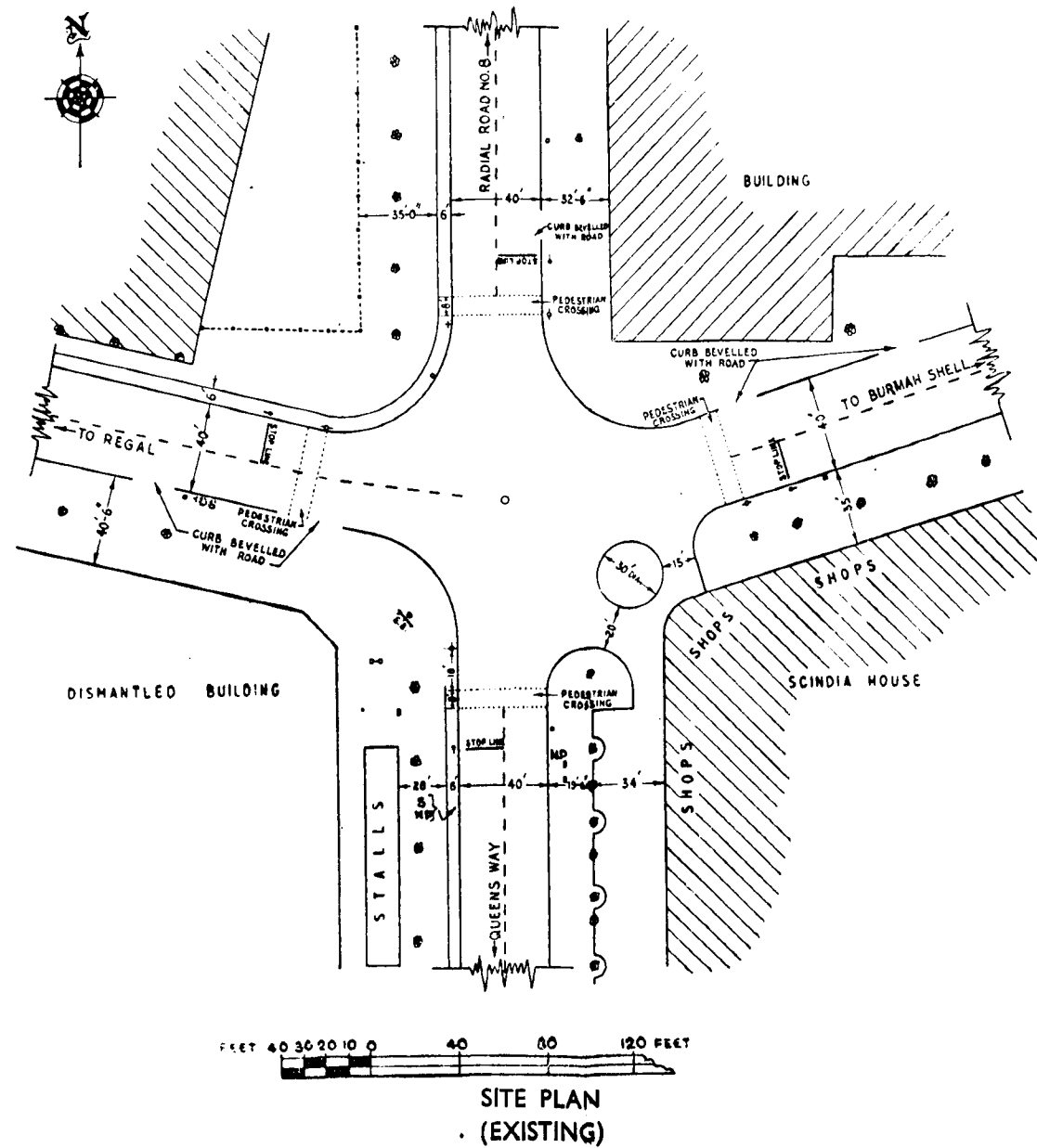
where D_v is the delay per vehicle

D_r is the total vehicle delay per period.

N is number of period per hour.

V is hourly volume of traffic.

PLATE I



SITE PLAN OF THE JUNCTION OF QUEENSWAY AND CONNAUGHT CIRCUS AND IMPROVEMENTS SUGGESTED.