INDIAN ROADS CONGRESS

NOTICES AND ANNOUNCEMENTS

NEW ADMISSIONS

LIST OF THE MEMBERS OF THE INDIAN ROADS CONGRESS ENROLLED FROM 21-6-1950 to 12-8-1950.

Serial No.	*Roll No.*	*Name*	*Address*
1375	1519	R. M. Parikh	Assistant Consulting Surveyor to the Government of Bombay, Town Planning & Valuation Department, Girdhar Buildings, Lal Darwaja, Ahmedabad.
1376	1520	S. S. Gill	Assistant Engineer, Fatehgar Sub-Division, Sirhind. (PEPSU).

DEATH

The Council have received with regret, the intimation of the following death :—

Serial No.	*Roll No.*	*Name*	*Address*
1	923	Shri Abdul Jabbar	Executive Engineer, Central Division, Patna.

1. INTRODUCTION.

1.1. The old Dufferin Bridge across the Ganga at Banaras provided for a single broad gauge track of the East Indian Railway and a single lane for the traffic on the Grand Trunk Road at the same level as the railway track. The bridge was opened to road traffic only at intervals as both the railway and the road traffic could not pass over it simultaneously. Every year, after the rains, a pontoon bridge was erected 1,200 feet upstream of the Dufferin Bridge to relieve congestion of the road traffic on the railway bridge. The pontoon bridge could not function throughout the year as the river level rose from 30 feet to 45 feet during the monsoon between July and October. During this period the pontoon bridge had to be dismantled and all road traffic was diverted to the Dufferin Bridge. This often meant long hours of waiting for such traffic.

1.2. As both railway and road traffic tended to increase rapidly from year to year, the problem of handling its ever increasing volume, especially during the monsoon, became very serious. The steel work of the old railway bridge had also deteriorated in the course of time and required replacement. Regirdering of the bridge thus became an indispensable necessity and, realising this fact, the Railway Board sanctioned a scheme in 1939 which provided for two-way road and rail traffic on separate decks at a cost of about Rs 80 lakhs. Of this sum, the U. P. Government agreed to contribute Rs 28 lakhs to cover the additional cost of providing a roadway over the bridge.

1.3. The work was, however, postponed on account of World War II and was actually started in October 1945. The reconstructed bridge provides for a double-line of railway and an independent roadway takes overhead. The entire work of regirdering was carried out by the Railway without stopping train traffic even for a day. The roadway consists of a 25-ft wide cement concrete carriageway for vehicular traffic to take I. R. C. Class AA Loading plus 3½ ft wide footpaths on either side for pedestrians.

1.4. The bridge was formally opened to railway traffic in December 1947 by the Hon'ble Pandit Govind Ballabh Pant, Premier of Uttar Pradesh, and renamed the 'Malaviya Bridge'. The work of completing the roadway was held up for various reasons. It is still not complete* in all respects but traffic was allowed to pass over it with certain restrictions from the 15th June 1949.

1.5. The regirdering of the Malaviya Bridge by the E. I. Railway engineers has received, as it deserves, countrywide appreciation. The work of constructing approaches to the bridge for road traffic by the Buildings and Roads Branch of the Public Works Department, Uttar Pradesh, although not so spectacular as the work done by the Railway, was nevertheless of considerable magnitude and importance and presented a number of interesting and difficult engineering problems. The scarcity of essential building materials such as steel and cement and the rising cost of manual labour had a vital effect on the design and method of executing the work.

* Written in August 1949.

2. STEAMER FERRY SERVICE.

2.1. The first problem which confronted the Department was the organization of a ferry service for road traffic during the monsoon since, although rail service was never interrupted during the regirdering operations, it was not possible to favour road traffic in the same way. In fact, when the E. I. Railway started regirdering work in October 1945, the bridge was closed to road traffic and remained closed altogether for three years, the period required for completing the regirdering. The Uttar Pradesh P. W. D., were thus faced with the problem of carrying all the Grand Trunk Road traffic across the mighty Ganga in flood by a free ferry service. The difficulties of the problem were enhanced by the following factors :—

(a) The volume of traffic was the largest ever ferried across a major river in the U. P.

An idea of the *daily* traffic crossing the Ganga at Banaras during the rains can be had from the following traffic census taken on the Dufferin Bridge shortly before the war brought heavy traffic on it :—

1.	Foot Passengers.	...	10,000
2.	Bullock carts.	...	150
3.	Motor cars and Lorries.	...	80
4.	Bicycles.	...	1,000
5.	*Ekkas* and *Tongas*.	..	100
6.	Cattle.	...	150

Motor traffic increased very rapidly during the war and was estimated at approximately 100 per cent more than these figures when the ferry service was inaugurated in 1945.

(b) The existence of the Dufferin Bridge near the site of the ferry created disturbances in the current of the river making navigation dangerous on the upstream side.

(c) There were rapid and frequent variations in the level of the river making it impossible to make any fixed arrangements for landing.

2.2. The steps taken to cope with the situation are described below:—

2.2.1. The ferry was run successfully during the monsoon of 1946 and 1947 with the following river craft:—

(i) Two steamers—S. S. Naraini & Ram Raj. The first was a double-decker surveyed for carrying 350 passengers and having overall dimensions of 90 ft by 12 ft by 24 ft. It was a wheel driven vessel with an engine of 65 H.P. and manned by a crew of seventeen. The S. S. Ram Raj was a light one-decker, surveyed for carrying 150 passengers with overall dimensions of 52 ft by 14 ft by 14 ft. This was a screw driven vessel with an engine of 32 H.P. and manned by a crew of eleven.

(ii) Two steel flats with overall dimensions of 60 ft by 16 ft by 14 ft each carrying 200 passengers and manned by six *manjis*, and

(iii) Several large size boats, 60 ft by 20 ft and five medium size boats, 35 ft by 15 ft each boat manned by six *manjis* and suitably planked to carry vehicular traffic.

2.2.2. Two steamers were obviously necessary for running the ferry regularly and efficiently. One steamer could always be relied on to be in action when the other broke down or was undergoing periodical servicing. The flats were attached to the steamers during the peak hours of traffic in the morning and evening. Vehicular traffic, including large trucks, was also taken on the boats which could be rowed across the river independently or attached to the steamers when the river was in flood.

2.2.3. During the rains of 1948, one of the steamers was replaced by a petrol tug of 300 H.P. run departmentally.

2.2.4. The site of the ferry, in relation to the sites of the pontoon bridge and the Dufferin Bridge, can be seen on the plan (Plate I). During the early and late stages of the monsoon season, when the current in the river was not very strong, the landing station on the Banaras side was located near the pontoon bridge (point E on Plate I) but, when the river was in flood, the landing station was shifted to the mouth of the Burna to avoid the risk of the steamer striking against a pier of the Dufferin Bridge.

2.2.5. The landing on the Moghalsarai side remained throughout at point C near the Dufferin Bridge on the downstream side. There was no suitable landing ground between the Dufferin Bridge and the Burna junction on the Banaras side on account of the height of the bank on that side. New ferry approach roads had to be constructed on both banks to join the ferry to the Grand Trunk Road.

2.2.6. A total rise of about 40 feet from the low water level of the river to its high flood level had to be provided for in the design. Apart from this, a sudden rise or fall in the level of the river, as much as 10 feet within 24 hours, was not an unusual happening. Any fixed arrangements for landing were, therefore, out of question. Parts of the pontoon bridge were used to provide approaches to the steamer and they served the purpose excellently. The position of the landing could be adjusted within a few hours as the level of the river varied. Even the heaviest trucks were carried to and from the boats with ease and speed.

2.2.7. The efficiency of the ferry service was universally commended by the travelling public who used it for three years.

3. ALIGNMENT OF APPROACHES.

3.1. The roadway on the main bridge has been raised 22 feet 6 inches above the old level, necessitating new approaches to the bridge, 2,005 feet long on the Banaras side and 2,510 feet long on the Moghalsarai side. A uniform gradient of 1 in 50 has been provided on both the approaches and, in order that this gradient should not be exceeded, a loop had to be provided on the Banaras side between the railway under-bridge and the Malaviya Bridge.

3.2. The road level at the under-bridge could not be raised as the overhead clearance could not be reduced below the existing minimum of 14 feet 6 inches. The old alignment had, therefore, to be abandoned and a new alignment adopted to obtain the required length of the road with a gradient of 1 in 50. The alignment on the Moghalsarai side follows the old embankment.

3.3. The approaches are 40 feet wide at formation level with a 25-ft B.T. (surfaced) carriageway and two $7\frac{1}{2}$ ft wide footpaths.

4. ABUTMENTS.

4.1. Masonry abutments, 22 feet high, have been constructed at the ends of the Malaviya Bridge to support the joists of the end "run-off" spans of the upper (highway) deck and to retain the earth filling behind (see Photo. No. 1). The problem of constructing heavy abutments was not an easy one as they were founded on fill of unreliable stability. In view of this, a load intensity of more than 0.8 ton per square foot on the base on either bank could not be permitted.

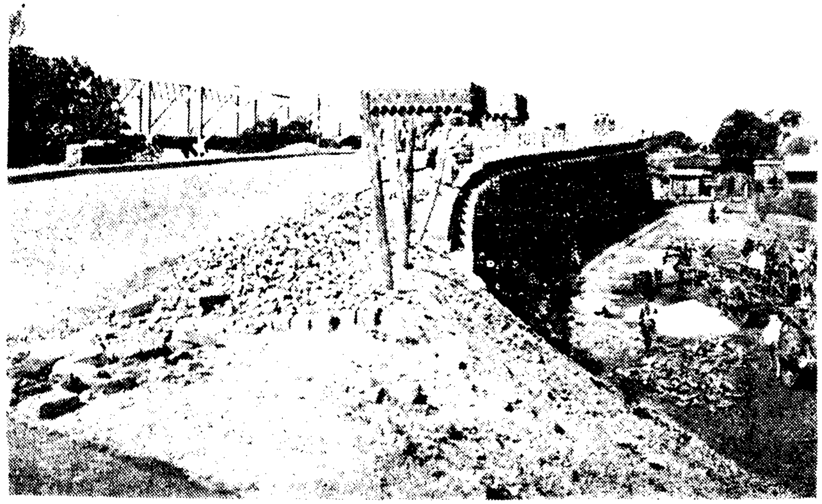
Photograph 1.
A View of the Moghalsarai Side Approach.

4.2. Different designs for the foundations were adopted on the two banks, the design of the superstructure being the same. The site on the Banaras side was full of debris and pile-driving to strengthen the base was out of question. A longitudinal crack, extending 10 feet below ground level, was also observed on this side and, therefore, a hollow-box type foundation was built to reduce the load on the base.

4.3. The abutment on the Moghalsarai side was built on 6 inch diameter *sal balli* piles spaced 1 foot 6 inches centre to centre and driven 15 to 20 feet below the top of old embankment (see Plate II). For calculations see Appendix III.

5. RETAINING WALL ON THE BANARAS SIDE.

5.1. A retaining wall, 300 feet long, has been built on the Banaras side, its height varying from 22 feet at one end to 16 feet at the other, to leave undisturbed the road to the railway goods-shed (see Photo. No. 2). The railway yard at the Kashi railway station is already congested and it was not considered advisable to reduce its space further by earthfilling. The retaining wall consists



Photograph 2.
The Kashi Side Approach.

of 6 bays of 50 feet each with a half inch clear space between the adjacent bays. The soil on which the wall stands is not firm as it (the soil) contains much debris from old ruins and every precaution had to be taken to safeguard the stability of the retaining wall. Construction in small bays of 50 feet length was adopted to localise any crack from subsidence of the foundation. It was originally contemplated to provide a reinforced concrete raft foundation to reduce the maximum intensity of pressure on the base but, on account of the shortage of cement and steel, the idea was abandoned. The retaining wall has, however, been suitably buttressed and the maximum intensity of the pressure on the base was thus reduced to about 0.8 ton per square foot. Details of the retaining wall are given in Plate III. The calculations are in Appendix III.

6. EARTHWORK ON THE MOGHALSARAI SIDE.

6.1. The earthwork on the Moghalsarai side was the critical test of success because of the following special problems:—

(a) The quantity of earth filling exceeded 75 lac cubic feet which was rather a large quantity for a short embankment of about half a mile.

(b) The old embankment was already 40 feet high near the gate of the Dufferin Bridge and it had to be raised further by 22 feet. Earth had to be taken from 10 feet deep borrow pits and thus the maximum lift involved was 70 ft. The average lift of earthwork worked out at about 25 feet.

(c) Earth was not available all along the embankment on both sides. Land for borrow pits was acquired permanently in four patches. Even then, the average lead came to about a furlong. Almost all the earth had to be taken from borrow pits on the north side as land on the south side, between the road and the railway embankments, was already very low and unsuitable for further excavation. Many borrow pits had to be abandoned because they yielded sandy soil.

(d) The pontoon bridge approach road which ran touching the toe of the embankment on the northside carried heavy traffic throughout the year. The work of conveying earth by headload was also hampered by fast traffic moving on the approach road. There were many accidents and ultimately the road had to be diverted to secure free access by the workmen to the embankment.

(e) On account of the great height of the embankment and the loose sandy nature of the soil used at places, considerable quantities of earth were washed away during the rains of 1947 and 1948.

(f) The wages of labour continued to rise during the progress of the work.

7. RATES.

7.1. Tenders for earthwork on the Moghalsarai side were originally invited in October 1946. The work was broken up into three groups to encourage competition among the contractors. The rates, at which the contract was finally settled, were as follows:—

Group.	Length (from the abutment.)	Approximate mean lead.	Approximate mean lift.	Rate.
I	460 ft	550 ft	28 ft	50/- per 1000 cu.ft.
II	660 ft	525 ft	23 ft	37/8/- do.
III	1450 ft	450 ft	14 ft	30/8/- do.

These rates were settled with one contractor on the basis of the lowest tendered rates. The quantity of earthwork being nearly equal in each of the three groups, the average rate for the whole work came to Rs 39 5/- per thousand cubic feet.

7.2. The work was started in February 1947 and about 15,00,000 cubic feet or approximately 20 per cent of the total was done up to July 1947. This was mainly in the first group. In July 1947, however, the contractor found that he was suffering a huge loss and, on the basis of the experience that he gained on this work, he put up his demand as follows:—

Group No.	Original rate	New Rate demanded by the Contractor
I	Rs 50-0-0 per 1000 cu. ft.	Rs 112 per 1000 cu. ft.
II	,, 37-8-0 do.	,, 102 do.
III	,, 30-8-0 do.	,, 73 do.

Thus the average rate demanded by the contractor worked out to Rs 95-11-0 as against Rs 39-5-0 per thousand cubic feet originally settled with him. This was obviously very high and fresh tenders were invited in November 1947. The lowest tender received was at Rs 70 per thousand cubic feet for the whole work. Even this was 78 per cent higher than the original rate.

7.3. As the contractor had taken approximately six months to carry out only 20 per cent of the work, it was considered doubtful whether it would be possible for him to finish the work within a reasonable time in spite of his being granted enhanced rates. At one stage it was seriously contemplated to abandon the earthwork for a length of 1,000 feet from the highest point and to carry the roadway on a sort of viaduct. The idea of a viaduct, however, had to be discarded owing to the scarcity of steel and cement.

8. DIRECT LABOUR.

8.1. Considering all the above points, the only alternative, though a very difficult one, was to do the work departmentally. The Chief Engineer finally ordered the work to be done by departmental labour and the results achieved confirmed the wisdom of this decision. The average rate for earthwork came to approximately Rs 52 per thousand cubic feet which was about 25 per-

8.4.6. The cost of earthwork done by lorries came to Rs 52 per thousand cubic feet. The details of the staff employed on the fleet of 18 lorries, the working cost of a lorry per day, and an analysis of the rate of Rs 52 per thousand cu. ft are given in Appendix I, which also gives the wages of labour prevalent at various stages of the work.

8.4.7. The lorries served a useful purpose in consolidating to some extent the freshly laid earth.

8.5. Work by Headload.

8.5.1. Work by headload was done in some portions of the embankment because, firstly, in certain conditions this agency was cheaper than lorries, secondly, there were not enough lorries to maintain the required progress of the work, nor enough working space for lorries even if they had been obtained somehow; and thirdly, the labour which became idle when lorries broke down was kept employed usefully in this way.

8.5.2. About 25 lakh cubic feet of earthwork was done by headload. The maximum strength of labour employed on head-load work was 600 and the average for 12 months was 300 per day.

8.6. Work by Donkeys.

8.6.1. A very useful contribution to the progress of the work was made by donkeys who could travel by routes difficult and risky for human labour. Only about 6 lakh cubic feet of earth-work was, however, done by this agency.

8.6.2. The maximum number of donkeys employed on the work was 120 and the average per day was 60 for six months. As it was very difficult to supervise their work, all earth moved by donkeys was paid for on a task basis at the same rates which were allowed to petty contractors for work by headload. The rates of earthwork by donkeys varied from Rs 40 to Rs 60 per 1000 cu. ft.

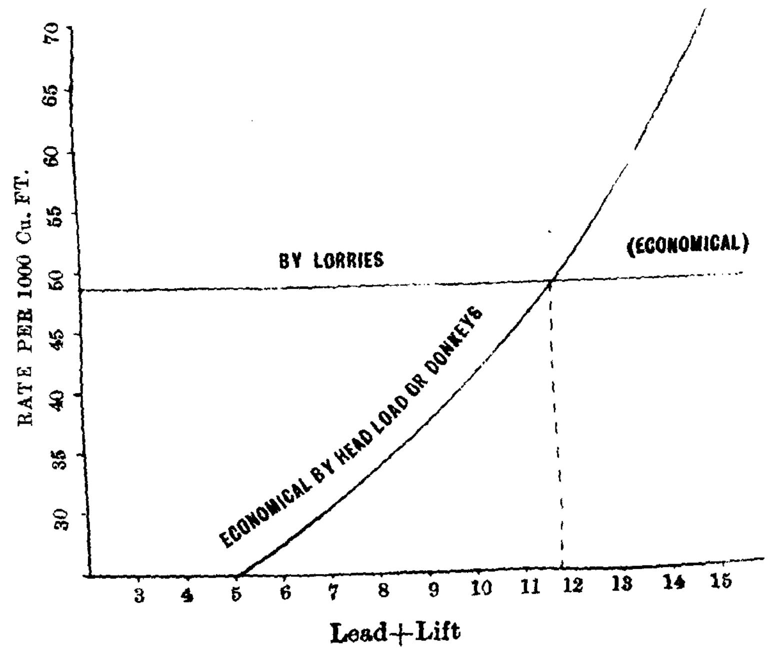
9. LORRIES *VS.* MANUAL LABOUR.

9.1. Looking at it from a general point of view, it will be seen that the labour and time taken for loading and unloading earth into lorries are the main factors in the cost of earthwork done by lorry. A slightly greater lead and lift will not make much difference to the cost and. therefore, for bigger leads and lifts, work by lorry will definitely be economical. For shorter leads and lifts, headloads should be economical.

9.2. The graph on the next page gives a comparative idea of the of earthwork by lorries and by headload. The abcissa indicates

lead plus lift, taking a lead of 100 feet to be equivalent to a lift of five feet. The ordinate shows the rate per 1000 cu. ft of earth-work. From this we find that up to an overall lead plus lift of say 12 units, work by headload is economical whereas beyond this, work by lorries is economical.

Graph Showing Rates of Earthwork by Lorries and Headload or Donkeys at Malaviya Bridge, BANARAS.



[100 Ft Lead=5 Ft Lift=One Unit]

9.3. The maximum rate of Rs 70 per 1000 cu. ft was incurred for work by headload involving a lead up to 600 feet and a lift up to 45 feet. The average rate for earthwork by headload was nearly the same as for earthwork by lorries, i.e., Rs 50 per 1000 cu. ft. This was accomplished by adjusting the range of operation of each type of agency in such a way that working did not go out of the economical limits indicated on the graph.

10. SOIL ANALYSIS.

10.1. No soil analysis was carried out before starting the work or during its execution but a few samples of soil have since been analysed. The results are given in Appendix II. It is a pity that accurate measurements of settlement were not taken during this period and a co-relation of settlement with the classification of soil is not possible now.

11. A USEFUL DEMONSTRATION.

11.1. The successful completion of earthwork by departmental labour has been hailed as "a most ceseful demonstration," of direct working when contractors begin to demand unreasonably high rates.

11.2. The high embankment on the Moghalsarai side was severely damaged during the rains of 1947 and 1948 while under construction. The problem of protecting it during the rains of 1949 after its completion caused anxiety. The permanent measure provided in the estimate is reinforced brickwork saucer drains but they could not be constructed as the sides of the embankment had still to settle and become stable. Empty drums of tar and bitumen cut into two were found very handy for temporary drains. Their top and bottom covers were removed and they were cut in two halves longitudinally and joined together by small (1 inch by ½ inch) bolts. Such drains were also constructed 100 feet apart to carry the rainwater from the drains on the top of the embankment. The tar drum drains have proved to be very effective and no serious cut of the embankment has so far (August 1949) been observed.

11.3. One and a quarter inch mild steel pipes fixed to reinforced concrete posts, 3 ft 6 inches high and 5 ft apart, will provide railings on the retaining wall on the Banaras side and on the embankment wherever it is 30 feet or more in height.

12. COST.

12.1. The entire work of constructing the approaches to the Malaviya Bridge has taken three years to complete at a cost of Rs 11 lakhs, including Rs 3½ lakhs for the free steamer ferry service during three monsoons.

Appendix I.

ANALYSIS OF RATES.

1. The following staff was employed on 18 lorries:—

					Rs.	A.	P.
(1) Transport Supdt.	1 No.	@	160	p.m.	160	0	0
(2) Unit Managers	3	,,	115	,,	345	0	0
(3) Unit Mechanics	3	,,	115	,,	345	0	0
(4) Asst. Mechanics	3	,,	65	,,	195	0	0
(5) Unit Cleaners	3	,,	60	,,	180	0	0
(6) Drivers	18	,,	64	,,	1,152	0	0
(7) Cleaners	18	,,	48	,,	864	0	0
					Rs 3,241	0	0

Expenditure on staff per lorry per day $= \frac{3,241}{30 \times 18} =$ Rs 6

2. Analysis for the minimum charge of Rs 20 per day per lorry:

(1) Staff (see above)	Rs 6	0	0	per day
(2) Petrol 2 gallons @ 2-4-0 for 13½ miles per day on average.	,, 4	8	0	do.
(3) Lubricant 1/8 gallon @ 5-0-0	,, 0	10	0	do.
(4) Changing tyres, etc.	,, 2	0	0	do.
(5) Depreciation 2 per cent of cost per month. i.e., $\frac{2}{100} \times \frac{10,000}{30}$ per day	,, 6	11	0	do.
	Rs 19	13	0	per day
	Say Rs 20	0	0	per day.

3. Analysis of cost of 1,000 cu. ft earthwork by lorries:

(1) Loading: 2 lorries making 9 trips per day and carrying 90 cu. ft in each trip carried 1,620 cu. ft of earth at a cost of Rs 35-4-0. Cost of loading 1,000 cu. ft	21	12	0
(2) Charge for lorry 1,000 cu. ft $= \frac{1,000 \times 20}{810}$	24	11	0
(3) Charge for idle days 20 per cent of above	5	0	0
(4) Maintenance of roadway for the lorries, watering, etc.	0	9	0
	Rs 52	0	0 per 1,000 cu. ft.

This rate of Rs 52 per 1,000 cu. ft for earthwork done by lorries was almost uniform irrespective of lead and lift.

4. Rates of Departmental Labour employed on work :

(1) Mate	Rs 1 4 0	per day
(2) Beldar or adult male labourer	1 0 0	do.
(3) Coolie (woman labourer)	0 10 0 to 0 12 0	do.

Almost within a week the above rates had to be increased by two annas per day for the mates and beldars and one anna per day for the coolies. After two months, when the harvesting season arrived, the rates had to be increased as detailed below:—

(1) Mate	Rs 1 8 0	per day
(2) Beldar (man)	,, 1 5 0	do.
(3) Coolie (woman)	,, 0 15 0	do.
(4) Boy or girl	,, 0 14 0	do.
(5) Donkey	,, 2 0 0	do.

Most of the work was carried out on the above rates of labour although the maximum rates paid on the work were:—

(1) Mate	Rs 1 10 0	per day
(2) Beldar (man)	,, 1 8 0	do.
(3) Coolie (woman)	,, 1 2 0	do.
(4) Boy or girl	,, 1 0 0	do.
(5) Donkey	,, 2 8 0	do.

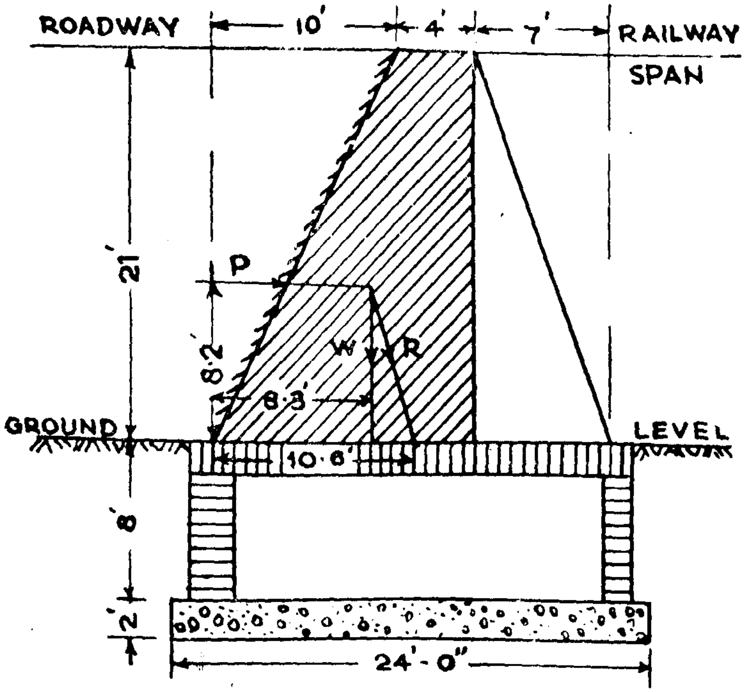
Appendix II.

SOIL ANALYSIS OF THE SAMPLES OF APPROACH ROAD TO MALAVIYA BRIDGE (MOGHALSARAI SIDE)

Serial No.	Location	Weight of wet soil + dish	Weight of dry soil + dish	Moisture	Liquid limit	Plastic limit	Plasticity Index	Sand Content	Sulphate Percentage	Optimum moisture	Remarks
1	Soil sample of Approach Road to Malaviya Bridge from Moghalsarai side.	$D_6 = 73.650$ gm	63.200 gm	10.450 gm							
	RIGHT	$D_{11} = 25.750$ gm	25.300 gm	0.450 gm	31.3	18.5%	12.8	30.10%	Below 0.10%	11.8%	
2	Soil sample of Approach Road to Malaviya Bridge from Moghalsarai side.	$D_8 = 69.550$ gm	60.200 gm	9.350 gm							
	LEFT	$D_{12} = 33.580$ gm	33.100 gm	0.480 gm	30.9%	18.7%	12.2	33 30%	Below 0.10%	11.4%	
3	Soil sample from the Borrow pit on	$D_3 = 65.580$ gm	58.400 gm	7.180 gm							
	RIGHT side	$D_4 = 30.600$ gm	30.300 gm	0.300 gm	22.7%	14.8%	7.9	52.80%	Below 0.10%	13.2%	

Appendix III.

(1) Calculations for Abutment on Kashi Side.



Height of abutment, h $= 21$ ft

Length of abutment $= 32$ ft 6 in.

Equivalent surcharge h_1 due to two 70-ton tanks per foot run of abutment

$$= \frac{140 \times 2240}{32.5 \times 21 \times 100}$$

$= 4.6$ ft of earth weighing 100 lb/cft.

Total earth pressure P at the back $= \frac{1}{2} \times \frac{1 - \sin.\phi}{1 + \sin.\phi} \times w \times (h + 2h_1) \times h$

$$= \frac{1}{2} \times \frac{1 - \frac{1}{2}}{1 + \frac{1}{2}} \times 100 \times (21 + 9.2) \times 21$$

$= 10{,}500$ lb.

Height of application of P from bottom

$$= \frac{h + 3h_1}{h + 2h_1} \times \frac{h}{3}$$

$$= \frac{21 + 3 \times 4.6}{21 + 2 \times 4.6} \times \frac{21}{3}$$

$= 8.2$ ft

Weight of earth filling at heel	$= \frac{1}{2} \times 10 \times 21 \times 100$	$= 10{,}500$ lb.
Weight of inner masonry	$= \frac{1}{2} \times 10 \times 21 \times 120$	$= 12{,}600$,,
Weight of masonry	$= 4 \times 21 \times 120$	$= 10{,}080$,,
Weight of outer masonry which exists for half the length of abutment only in alternate buttresses	$= \frac{1}{2} \times \frac{1}{2} \times 21 \times 7 \times 120$	$= 4{,}410$,,
Total weight W of the abutment per foot run		$= 37{,}590$,,

Point of application of W from the heel

$$= \frac{\frac{1}{3} \times 10 \times 10500 + \frac{2}{3} \times 10 \times 12600 + 12 \times 10080 + 16.3 \times 4410 \text{ ft.}}{37590}$$

$= 8.3$ ft

The distance of resultant R from the heel

$$= 8.3 + 8.2 \times \frac{P}{W}$$

$$= 8.3 + 8.2 \times \frac{10500}{37590}$$

$= 10.6$ ft i.e., the resultant passes very nearly through the middle of base.

Load on Foundation.

Weight of foundation concrete $= 36 \times 24 \times 2 \times 130$

$= 224{,}640$ lb.

Weight of hollow masonry box approximately $= (32.5 \times 22 \times 8 - 6 \times 3.5 \times 6 \times 20) \times 120$ lb.

$= 384{,}000$ lb.

Weight of abutment $= 37590 \times 32.5 = 1,222,000$ lb.

Weight of earth filling at sides $= 2 \times 24 \times 1\frac{3}{4} \times (8 + \frac{21}{2}) \times 100$
$= 155,400$ lb.

Weight of earth filling at the back $= 36 \times 1 \times 29 \times 100$
$= 104,400$ lb.

Weight of earth filling at the front $= 36 \times 1 \times 8 \times 100$
$= 28,800$ lb.

Weight of roadway $= 32.5 \times 15 \times 110$ approximately $= 53,600$ lb.

Weight of roadway over the Railway span $= 32.5 \times 10 \times 150$ approximately, span being 20 ft.
$= 48,800$ lb.

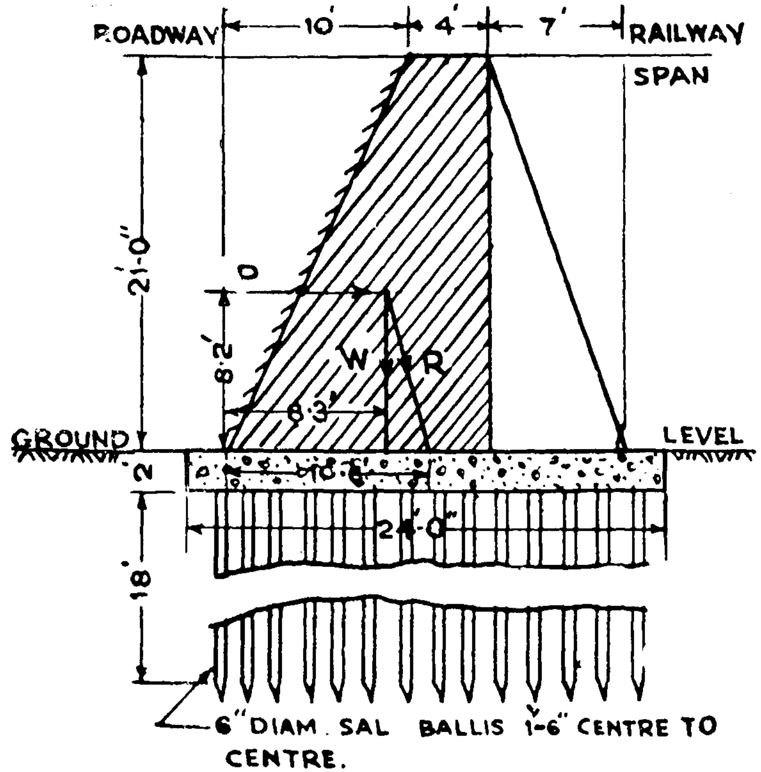
Weight of surcharge due to two 70-ton tanks $= 32.5 \times 15 \times 4.6 \times 100$
$= 224,200$ lb.

Total weight of foundation $= 2,445,800$ lb.

Weight on foundation per sq. ft, 10 ft below the ground level $= \frac{2,445,800}{36 \times 24}$ lb.
$= 2,831$ lb.

Equivalent weight per sq. ft at the ground level $= 2831 - 10 \times 100$ lb.
$= 1,831$ lb.
$= 0.82$ ton—which is very nearly equal to 0·8 ton per sq. ft., the safe bearing pressure of soil.

(2) Calculations for Abutment on Moghalsarai side.



Abutment on Moghalsarai side has the same superstructure as on the Kashi side but the foundation consists of 2 feet cement concrete over 6 inches diameter *sal-balli* piles, 18 ft long and driven 1 ft 6 inches centre to centre.

Load on Foundation

Weight of foundation concrete	$= 36 \times 24 \times 2 \times 130$	$=$ 224,600 lb.
Weight of abutment	$= 37590 \times 32.5$	$=$ 1,222,000 ,,
Weight of earthfilling at the sides	$= 2 \times 24 \times 1\frac{3}{4} \times \frac{21}{2} \times 100$	$=$ 88,200 lb.
Weight of earthfilling at back	$= 36 \times 1.5 \times 21 \times 100$	$=$ 113,400 ,,
Weight of Roadway at back	$= 32.5 \times 15 \times 110$	$=$ 53,600 ,,
Weight of Roadway over railway span, approximately		$=$ 48,800 ,,
Weight due to surcharge of tanks	$= 32.5 \times 15 \times 4.6 \times 100$	$=$ 224,200 ,,
Total weight on foundation		$=$ **1,974,800 lb**

Weight on the base per sq. ft $= \frac{1974800}{36 \times 24}$ lb.

$= 2{,}286$ lb.

Equivalent weight per sq. ft at the ground level $= 2286 - 2 \times 100$

$= 2086 \text{ lb} = \cdot 93$ ton.

As the permissible load on the soil was only 0.8 ton per sq. ft, 6 in. *sal balli* piles, 18 ft long and 1 ft 6 in. centre to centre, were driven to take up the load.

Load on one pile $= \frac{3}{2} \times \frac{3}{2} \times 1 = 2.25$ tons

Safe load on each pile being $= P$

$$= \frac{2\,w\,h}{s+1} \text{ tons}$$

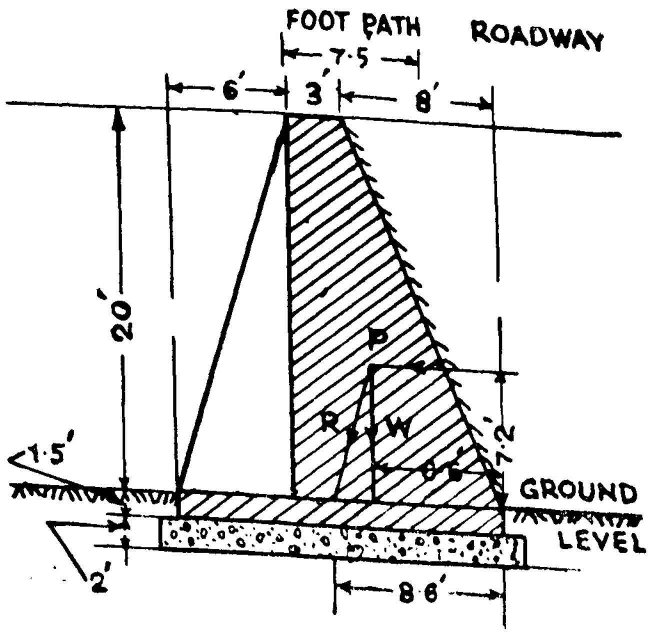
where w = weight of monkey in tons $= 0.2$ ton

h = fall of monkey in ft $= 10$ ft

s = penetration of pile in the last blow $= \frac{3}{8}$ inch

$P = \frac{2 \times .2 \times 10}{\frac{3}{8}+1}$ $= 2.9$ tons which is safe

(3) Calculations for Retaining Wall on Kashi side near the abutment end.



Retaining Wall on Kashi side 300 ft in all was constructed in panels of 50 ft each, maximum height near the abutment being 0.7 ft and minimum height at the end being 14.7 ft.

Average height of retaining wall in the 1st panel, h $= 20.2$ ft say 20 ft

Length of retaining wall $= 50'$

Equivalent surcharge h_1 due to only one tank $= \frac{70 \times 2240}{50 \times 20 \times 100}$

$= 1.6$ ft say 2 ft as the other tank of earth weighing 100 lb/cft. will be at a distance of more than 20 ft from the retaining wall and will exert little pressure, 20 ft being the height of the wall. The retaining wall carries a raised foot path of 7 ft - 6 in. width and the roadway is behind it.

Total earth pressure at the back

$$= \frac{1}{2} \times \frac{1 - \sin\phi}{1 + \sin\phi} \times w \times (h + 2h_1) \times h$$

$$= \frac{1}{2} \times \frac{1 - \frac{1}{2}}{1 + \frac{1}{2}} \times 100 \times (20 + 2 \times 2) \times 20 \text{ lb.}$$

$$= 8{,}000 \text{ lb.}$$

Point of application of P from bottom

$$= \frac{h + 3h_1}{h + 2h_1} \times \frac{h}{3}$$

$$= \frac{20+6}{20+4} \times \frac{20}{3}$$

$$= 7.2 \text{ ft.}$$

Weight of earth at heel	$= \frac{1}{2} \times 8 \times 20 \times 100 =$	8,000 lb.
Weight of inner masonry	$= \frac{1}{2} \times 8 \times 20 \times 120 =$	9,600 lb.
Weight of masonry	$= 3 \times 20 \times 120 =$	7,200 lb.
Weight of Outer masonry	$= \frac{1}{2} \times 20 \times \frac{6}{2} \times 120 =$	3,600 lb.

which exists for half the length of the retaining wall in form of buttresses.

Total weight W of the retaining Wall per ft run $= 28{,}400$ lb.

Point of application of W from the heel

$$=\frac{8000\times\frac{8}{3}+9600\times\frac{2\times8}{3}+7200\times9\frac{1}{2}+3600\times13}{28400}$$

$=6.6$ ft.

The distance of resultant R from the heel

$=6.6+\frac{P}{W}\times7.2=6.6+\frac{8000\times7.2}{28400}=8.6$ ft. i.e , the resultant passes very nearly through the middle of base.

Weight on foundation.

Weight of foundation concrete	$=19\times2\times130$	$=$ 4,940 lb.
„ masonry slip	$=17\times1\frac{1}{2}\times120$	$=$ 3,060 „
„ retaining wall per ft		$=$28,400 „
„ earth filling at back & front $1\times(21\frac{1}{2}+1\frac{1}{2})\times100$	$=23\times100$	$=$ 2,300 „
Equivalent weight due to surcharge of tank	$=12\times2\times100$	$=$ 2,400 „
Total weight on foundation per ft		$=$41,100 „

Weight on foundation per S. ft. 3.5 ft below ground level $=\frac{41,100}{19}$

$=$ 2,163 lb.

Equivalent weight at the ground level per S. ft $=$ 2,163 – 350 lb.

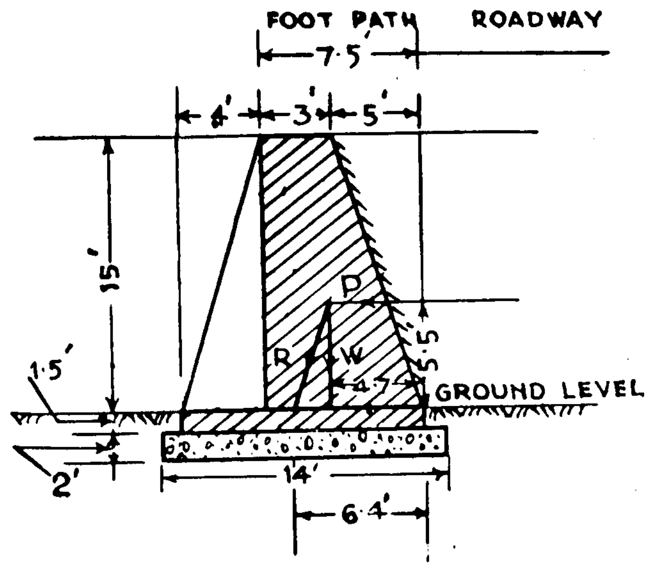
$=$ .81 ton.

which is very nearly equal to .8 ton per S. ft permissible on the soil.

(4) Calculations for the Retaining Wall at the end away from abutment.

Average height of Retaining Wall in the last panel $=15.2$ ft say 15 ft

Length of retaining wall. $=50$ ft



Equivalent surcharge h_1 due to one tank of 70 tons weight. $=\frac{70\times2240}{50\times15\times100}=2.1$ ft

$=$ Say 2 ft of earth weighing 100 lbs/cft.

Total pressure at the back

$$P=\frac{1}{2}\times\frac{1-\sin\phi}{1+\sin\phi}\times W\times(h+2h_1)\times h$$

$$=\frac{1}{2}\times\frac{1-\frac{1}{2}}{1+\frac{1}{2}}\times100\times(15+2\times2)\times15 \text{ lb.}$$

$=4750$ lb.

Point of application of P from bottom

$$=\frac{h+3h_1}{h+2h_1}\times\frac{h}{3}=\frac{15+3\times2}{15+2\times2}\times\frac{15}{3}$$

$=5.5$ ft.

Weight of earth at heel	$=\frac{1}{2}\times5\times15\times100=$	3750 lb.
„ „ inner masonry	$=\frac{1}{2}\times5\times15\times120=$	4500 lb.
„ „ masonry	$=3\times15\times120$	$=$ 5400 lb.
„ „ outer masonry	$=\frac{1}{2}\times15\times\frac{4}{2}\times120=$	1800 lb.
Total weight W of the retaining Wall per ft run		$=$ 15450 lb.

Point of application of W from the heel

$$=\frac{3750\times\frac{5}{3}+4500\times\frac{2\times5}{3}+5400\times6\tfrac{1}{2}+1800\times9\tfrac{1}{3}}{15450}$$

$=4.7$ ft.

Distance of resultant R from the heel

$$=4.7+5.5\times\frac{4750}{15450}\text{ ft.}$$

$=6.4$ ft. *i.e.*, the resultant passes through the middle third of the base with an eccentricity of .4 ft from centre.

Weight on foundation.

Weight of foundation concrete	$=14\times2\times130$	$=$ 3640 lb.
,, ,, masonry slip	$=12\times1\tfrac{1}{2}\times120$	$=$ 2160 ,,
,, ,, retaining wall per ft.		$=$ 15450 ,,
,, ,, earth filling at back & front	$=(16\tfrac{1}{2}+1\tfrac{1}{2})\times100$	$=$ 1800 ,,
Equivalent load due to surcharge of tank	$=9\times2\times100$	$=$ 1800 ,,
Total weight on foundation		$=$ 24850 lb.

Weight on foundation per S. ft 3.5 ft below ground level $=\frac{24850}{14}$,,

$=1775$,,

Equivalent weight per S. ft. at the ground level $=1775-350$,,

$=1425$,,

Maximum weight per S. ft. due to eccentricity of the resultant

$$=\frac{W}{b}\left(1+\frac{6e}{b}\right)$$ where $W=$ total weight.

$b=$ base.

$e=$ eccentricity from the centre.

$$=1425\left(1+\frac{6\times.4}{12}\right)$$

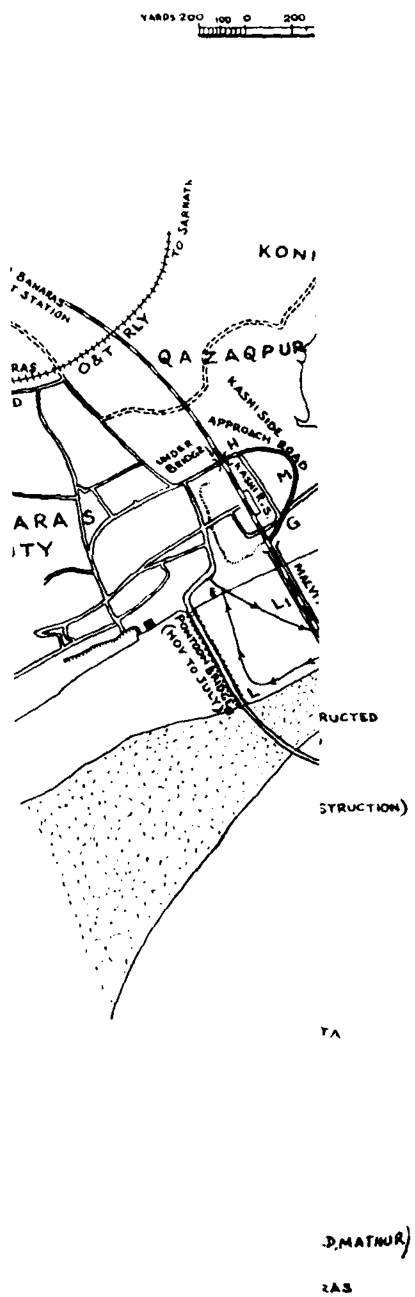
$=1710$ lb.

$=0.76$ ton which is very nearly equal to .8 ton/S. ft. permissible on the soil.

PLAN SHOWING COUR

PLATE I

MALVIYA



(ii) the Spoke-type in which twelve spokes are employed all notched into the hub but not passing through it as in the Arm-type.

In addition to these two types, two other types are also in use but on a small scale. They are :—

(i) a combination of the Spoke and the Arm types in use mostly in the sandy deserts of Rajasthan, and

(ii) a solid wheel of small diameter mostly in use in quarries for carting broken stones from the quarry to the road.

The combination wheel in Rajasthan is essentially a Spoke-type wheel whose iron tyre has been replaced with a wooden tyre about 3 inches to 5 inches wide.

SURVEY OF BULLOCK-CART WHEELS IN INDIA.

5. To ascertain the extent to which the Arm and Spoke type wheels are in use and the details of their construction, a survey was carried out throughout India by selecting a number of as nearly as possible equidistant stations, barring thinly populated areas, situated at about 100 to 150 miles from one another. Information obtained has been plotted on a map of India (Plate I).

6. Mr. Vesugar estimated the number of carts in pre-partition India at about 87 lakhs. The Author estimates that out of this number about 8½ lakhs are now in Pakistan :—

(1) East Bengal and Chittagong (assumed at 60 per cent of pre-partition Bengal)	5.17 lakhs
(2) West Punjab (assumed at 60 per cent of pre-partition Punjab)	
(3) Sind	2.14 ,,
(4) North West Frontier Province	0.90 ,,
(5) Baluchistan (assumed at the same figure as at N.W.F.P.)	0.07 ,,
	0.07 ,,
	8.35
	Say 8½ lakhs.

TYPES OF WHEELS AND THEIR NUMBER.

7. It will be seen from Plate I that the Arm-type wheel operates in the Punjab, Uttar Pradesh, most of Bihar, West Bengal and some parts of Orissa. According to Mr. Vesugar's figures its number may be estimated at about 26½ lakhs, including contiguous

territory formerly known as Indian States. The Spoke-type wheel operates in the rest of the country, barring a small area near Kutch. Its number may be estimated at about 52 lakhs. As Mr. Vesugar's figures were Province by Province while the regions of the two types of wheels cover portions of different Provinces (now States) it is not possible to form a more correct estimate of the number of these types than indicated above.

ARM-TYPE WHEEL TO BE RETAINED.

8. In the previous Papers of this series it was suggested that the arm-type wheel should be replaced with the spoke-type. This census has disclosed that this is not practicable as the number of carts employing arm-type wheels is very large and it is too much to expect the village wheel-wrights to change over from one type to the other. Moreover, it is possible to get over the most unsatisfactory feature of the arm-type wheel, viz., an unevenly wearing wooden tyre, which damages road surfaces and is more expensive to maintain, by providing a steel tyre. Hence, the Author has come to the conclusion that the arm-type wheel should be retained and improved to the extent required.

VARIATIONS IN SCANTLINGS.

9. The method of manufacturing each type of wheel is the same at all stations at which it is employed, but there are a number of variations in the sizes of the components, viz., tyre, felloe, spokes and hub. However, it must be mentioned to the credit of the wheel-wright that the basic structural stability of the design of each type is the same everywhere. For instance, if a spoke of a different size is employed at any station, the sizes of the hub and felloe are also altered to provide practically the same embedment for the spoke as at any other station. The phenomenon is interesting since, although the wheel-wright cannot explain why he uses particular sizes, he has arrived at what may be called a correct mathematical design by his hit and miss methods and age-old experience.

10. The Author has no hesitation in heartily endorsing the well-merited tribute which Mr. Murrell once paid to the ingenuity of the wheel-wright.

In the tests carried out during this investigation, the Author has not come across any other type of wheel sent for testing which is comparable to the present wheel in respect of structural stability, ease of manufacture, and simplicity of design. What the wheel-wright needs to be taught is the employment of standard sizes, for the tyre, felloe, spokes, and hubs, for use at every station throughout India.

VARIATIONS IN WHEEL-DIAMETERS.

11. The most confusing variations are in wheel.diameters which range from about 3 ft to about 6 ft in both types of wheels entailing a variation from about 68 lb to 114 lb per ton in the tractive effort and they seem to have no relation to the heights of bullocks employed. Since, for the same width of tyre, wheel-diameter governs the tractive effort of a wheel and the pay-load a cart can carry, it is necessary to find, if possible, reasons for the variations in wheel-diameters and the extent to which it is possible to eliminate them.

WHEEL-DIAMETERS VIS-A-VIS BULLOCKS' HEIGHTS.

12 For ease of traction a cart should remain parallel to the road surface at all times as in the tests on tractive effort. It also seems desirable that the pull should be exerted at the same height above the road level as that of bullocks' necks, although in the case of four wheeled vehicles (e.g., four-wheeled carts in and around Delhi, camel carts in Sind, heavy duty godown carts in Bombay and all four-wheeled horse-drawn vehicles in almost all countries) the direction of the pull is slanting up from the point of application on the vehicle to the point of exertion on the draught animal's body. The reason for this appears to be two-fold :—

(i) Ease of loading for which a low platform is more convenient.

(ii) Cost, which for four wheels, is greater than for two.

13 Ease in loading and unloading will have to be considered in the case of bullock-carts also, but as they employ only two wheels, the cost factor is not as important as in the case of four-wheeled vehicles. The diameter of their wheels should, as far as possible, be governed by the height of bullocks' necks, consistently with ease of loading.

14. With the scantlings normally employed in the construction of bullock-carts, the wheel-diameter in relation to the height of bullocks' necks (see Fig. 1) should work out as follows:—

$D = 2\,(H - 13 \text{ in.})$, where

D = wheel diameter

H = height of bullocks' necks.

The allowance of 13 in. made in this formula is on the assumption that a straight scantling is used as the yoke-piece as in the South of India. In practice, various expedients are employed at different stations. In Bombay a curved scantling is used and,

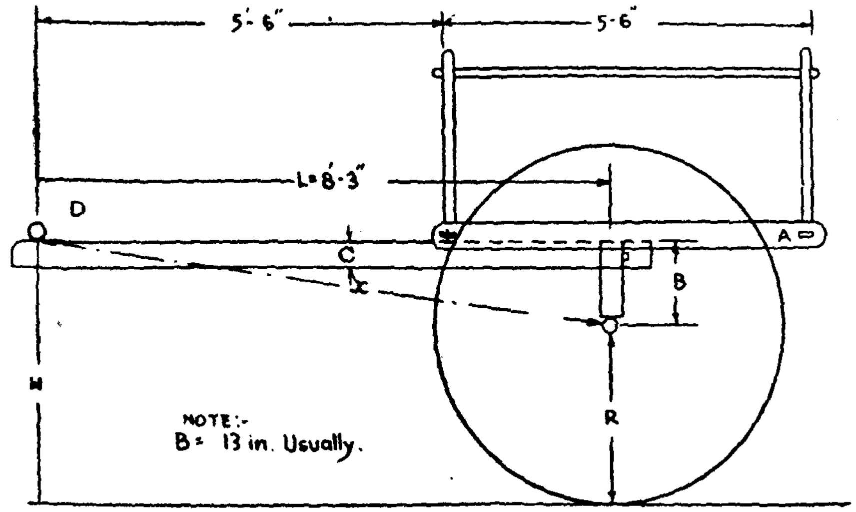


Fig. 1.

at many up-country stations, two round yoke-pieces sloping from the yoke to the axle are employed. However, these variations do not affect the assumption if the cart has to remain at the same height as bullocks' necks.

15. The position of wheel-diameters in use at present as against the diameters which should be employed according to the above formula has been shown on the accompanying map of India (Plate I) in seven zones made up as follows:—

Spoke-type Wheels:

Zone 1 in which D is equal to or greater than $2\,(H - 13 \text{ in.})$

„ 2 „ „ D is up to about 12 in. less „ „

„ 3 „ „ D is 12 in. to 24 in. „ „ „

„ 4 „ „ D is 36 in. to 48 in. „ „ „

Arm-type Wheels:

Zone 5 in which D is 15 in. to 30 in. less than $2\,(H - 13 \text{ in.})$

„ 6 „ „ D is 15 in. „ „ „

„ 7 „ „ D is 15 in. to 30 in. „ „ „

A statement showing particulars of these variations station by station in each zone is enclosed (Annexure I).

REASONS FOR VARIATIONS IN WHEEL-DIAMETERS.

16. It will be seen from Annexure I that the present wheel-diameters correspond more or less to the heights of bullocks' necks in Zone 1 only. In all other zones they are *less* than what they should be by 4 in. to as much as 48 in. in the spoke-type wheel and from 6 in. to as much as 30 in. in the arm-type wheel. These are considerable variations for which there must be reasons. In the Author's opinion some of them may be as follows:—

(i) Variations in the height of bullocks within a few hundred miles in each zone (see Plate I) (in the city of Bombay the variation is 5 in.) are due probably to the import of bulls from other stations or zones. This is understood to be a common practice among professional carters. As a consequence, station to station variations in wheel-diameters in each zone are bound to occur.

(ii) Variation in the amount of work for carts in various zones and also at various stations in each zone. It is natural that a carter should adjust his expenditure on his cart in proportion to the work offering and adjust it on the various parts of his cart including the wheels.

(iii) Bullocks of the same height do not have the same stamina at every station. This also is natural since the feed a carter can offer to his animals will depend on what he can get locally and how much he can afford to spend on it. This latter will depend on the amount of work offering.

(iv) Number of bullocks available per cart in a region. If this is not the same in all regions then, at stations where the number of bullocks per cart is more, there will be less work per pair of bullocks.

(v) Climatic conditions may also have something to do with wheel-diameters, e.g., it may be that in the hot dry climate of the Rajasthan desert large wheels made of thin sections employing local wood may be more susceptible to warping than small wheels of thick sections as are commonly employed from Ahmedabad to Bikaner.

(vi) Influence of neighbouring district. This can be noticed at all stations on the border lines of these zones and most particularly in Saurashtra.

17. Unfortunately, sufficient data is not available on all these points but some idea on points (ii) and (iv) may be had from Mr. Vesugar's Paper (I.R.C. Paper No. J.—1943). It is interesting to note in the case of spoke-type wheels that Zone 1, comprising most of the Madras State and Mysore with a load of about 199 maunds per cart, finds it necessary to employ large wheel diameters, while Zone 2, comprising most of the Bombay State and Hyderabad with a load of about 148 maunds per cart, employs smaller wheel-diameters. Zone 3, made up for the most part of Madhya Pradesh with a load of only about 93 maunds per cart, employs still smaller wheel-diameters, while Zone 4, comprising most of the desert part of Rajasthan with not much produce, employs the smallest wheel diameters. The probable reason why the arm-type wheels in Zones 5 and 6, with a high load per cart, are smaller in diameter than those in Zone 1 may be:

(i) With their broad tyres their tractive effort is less and, therefore, in spite of their smaller diameters, they may be capable of pulling greater loads.

(ii) The number of bullocks per cart in these Zones (Punjab 12; U.P., Bihar, Orissa, and Bengal, about 9 each) being greater than in Zone 1 (Madras 4·6), each pair of bullocks has less work to cope with without tiring even if it entails a greater tractive effort on account of a smaller wheel.

That a broad tyre requires less tractive effort has been established by our tests also and here is another proof of the wheel wright's instinct in finding from experience what suits his requirements best.

18. We have test data on the tractive effort of wheels with a 2 in. tyre in all ranges of wheel-diameters. From this it is possible to have an idea of the tractive effort of the average wheels employed in Zones 1, 2 and 3 which employ 2 in. tyres and their comparison with the work offering. This works out as follows:—

Zone	Average diameter in use	Tractive effort of wheel	Produce per cart
No. 1	61 in.	72 lb/ ton	199 md.
No. 2	53 in.	78 lb/ ton	148 md.
No. 3	46 in.	86 lb/ ton	93 md.

The variation in the tractive effort in these Zones is of the order of about 10 per cent while the variation in the amount of work offering is far greater. Strictly, in proportion to the work offering, the wheel-diameters in these zones and their tractive efforts should work out as follows :—

Zone	Produce per cart	Tractive effort	Wheel-diameters
No. 1	200 md.	72 lb/ ton	61 in.
No. 2	150 md.	95 lb/ ton	42 in.
No. 3	100 md.	144 lb/ ton	30 in.

That is, on the assumption that the average work offering in Zones 2 and 3 has been distributed throughout the year to the same extent as in Zone 1, the wheel-diameters employed therein should work out as shown above. The reason why they are bigger in Zones 2 and 3 may probably be that the distribution of work in these zones may not be the same as in Zone 1.

19. That is the work offering in these Zones may be confined to fewer months in a year than in Zone 1 and, therefore, the work offering in these months may be greater than what their average produce per annum indicates. This will necessitate larger wheel-diameters. The fact that there is one monsoon and therefore one major crop in a year in these zones and two monsoons and, therefore, two major crops in a year in Zone 1 lends support to this view. Discussing the variation in the number of carts per 1,000 tons of produce (which is the same thing as the average load per cart) Mr. Vesugar arrives at the following conclusions :—

> "The difference can, therefore, be explained only by assuming that in certain regions carts carry more and/or work for greater number of days in a year than in others. Such an inference seems to be supported by the great variation in the number of bullocks per cart."

20. It was first intended to ascertain in this survey the gross load per cart including the pay-load at every station but it was found that weigh-bridges were not available at many stations and hence the information received would be more or less a guess. Secondly, the actual load carried per trip has only a limited

bearing on this point since it was necessary to know how many days in a year each cart worked and this, of course, it was impossible to know correctly. Finally, the load actually carried by a cart at the time of the survey was not likely to be the average.

21. It thus appears that the wheel-wright must have struck a balance between the work offering, its frequency, and the capacity and number of animals, and evolved a wheel which was economical in the circumstances. This appears a logical guess since, in the case of a motor lorry, it is possible to design the body to carry a certain load and then provide an engine, powered suitably to pull it, while, in the case of a cart, the bullock and the chassis are there and the only item which can be adjusted according to work, is the wheel.

VARIATIONS IN WHEEL-DIAMETERS INEVITABLE.

22. It would thus appear that there *are* reasons, economic, climatic, or local, which have not made it possible for the wheel-wright to employ wheel-diameters corresponding to the height of bullocks and for want of fuller data on these reasons it would be **impossible for us to standardize wheel-diameters on any ground other than the present practice.**

PROPOSED WHEEL-DIAMETERS.

23. The main reason, of course, for a wheel-wright's preference for a small wheel-diameter is that it is cheaper and, where either there is insufficient work per cart, or the bulls are strong, or climatic conditions are unfavourable, or a wide tyre is employed, the carter employs a wheel with a smaller diameter. Therefore, the Author suggests that **in the proposed wheels of 3½ in. tyre it is best to stick to the same principle, viz., that the cost of the new wheels may as far as practicable be the same as that of the present wheel, provided that it does not entail a greater tractive effort than the present wheel.**

24. Wheels of 3½ in. tyre which can be manufactured within the cost of wheels with 2 in. tyre at present in use, exclusive of the axle-equipment, have been mentioned in column 2, Appendix C, Annexure II Part IV of this Series.* The relative tractive effort of wheels with these two tyres, with or without provision for self-alignment and with smooth cast iron or uneven mild steel bearings, has been shown in chart 2, Appendix E, Annexure IV of

* Paper No. 148 printed at pages 65 to 153 of the Journal of the Indian Roads Congress, Vol. XV-Part 1.

the same Paper. Combining these two tables the following indication can be had of the diameters to be employed in the proposed wheels on the basis mentioned in para. 23.

Wheel-diameters with 3½ in. Tyre.

Present range of wheel-diameters	Present diameter (2 in. tyre) inches	Diameter for same tractive effort as (2) (3½ in. tyre) inches	Diameter for same cost as (2) (3½ in. tyre) inches	Suggested standard (3½ in. tyre) inches
1	2	3	4	5
3 ft to 3 ft 6 in.	36 39 42	26 30¼ 34	36	35
3 ft 6 in. to 4 ft	45 48	36¾ 40	39	40
4 ft to 4 ft 6 in.	51 54	43 45¾	45	45
4 ft 6 in. to 5 ft	57 60	48 49½	51	50
5 ft to 5 ft 6 in.	63 66	50½ 51¾	57	52 to 57
5 ft 6 in. to 6 ft	69 72	52½ 53	63	53 to 63

Note: (1) The tractive effort of the proposed wheels for the diameters in col. 5 will be the same as that of corresponding wheels in use at present if they have a provision for self-alignment.

(2) It will, on an average, be about 8 lb less if they have no provision for self-alignment but have lathe turned axles and smooth cast iron bearings.

(3) It will be about 2 lb less if they have no provision for self-alignment but have hand-made axles and mild steel bearings as at present.

25. In practice it may be useful to round these figures somewhat on the lines indicated below, State by State, to make it easy for the Administrations concerned to enforce and regulate the dimensions from time to time till the wheel-wrights and carters get used to them:—

Administration	Present wheels	Proposed wheels
1. Saurashtra, Cutch, Rajasthan, the Gwalior district of Madhya Bharat, Vindya Pradesh and Madhya Pradesh.	3 ft to 4 ft	3 ft to 3 ft 3 in.
2. Punjab, PEPSU, Hima-Chal Pradesh and Uttar Pradesh.	4 ft to 5 ft	3 ft 3 in. to 4 ft 3 in.
3. Orissa, Madras, Mysore, Coorg and Travancore-Cochin.	5 ft to 6 ft*	4 ft 3 in. to 5 ft 3 in.
4. Bombay, Hyderabad, the Indore district of Madhya Bharat, Bhopal, Assam, Bihar and Bengal.	4 ft to 4 ft 6 in.	3 ft 3 in. to 3 ft 9 in.

PROPOSED TYRE-WIDTH.

26. These limits will allow the wheel-wrights some latitude to select a wheel-diameter in each range which will entail the same cost of construction as at present, exclusive of the axle-equipment. In addition, administrations can be told, if necessary, the wheel-diameters to be employed in each district under their jurisdiction with a suitable variation to suit local conditions. All wheels throughout India will have the same tyre, i.e., 3½ in. whether they are of the arm or spoke type, the wheel-wright being free to choose the type according to his present practice. The axle equipment, so long as it is the same as at present, will continue to be manufactured by the wheel-wrights but the hub will have to be made to a design which would permit a self-aligning device being incorporated into it later without the necessity of altering the hub or the wheel in any way. A few stations like Ahmedabad, Jodhpur, Jaipur, Agra, Bikaner, and Allahabad, which at present employ wooden tyres wider than 3½ in., may be

* Carts of 6 ft wheel diameter seem to operate only in Mysore with a total of some 34,000 carts out of 78½ lakhs for India.

allowed to adopt any width in excess of 3½ in. so long as a ⅜ in. thick steel tyre is used. **Wooden tyres, either not covered with a steel tyre or covered with a steel tyre narrower than the felloe (as employed at present in the urban areas of Zones 5 and 6) will have to be banned.**

REDUCED WHEEL-DIAMETERS.

27. **The basis of these recommendations is the employment of a broad tyre to the extent it is feasible to do so without exceeding either the cost (excluding axle-equipment or the tractive effort or the method of manufacturing the present wheels.** But a broad tyre increases the cost and, therefore, to conform to the present cost, the wheel-diameters will have to be necessarily reduced. Will these reduced wheel-diameters incline the cart and entail any hardship on the bullocks? Will they also entail a greater tractive effort on account of this incline? Finally, will the carters object to reduced wheel-diameters?

INCLINE OF THE CART.

28. It has been shown above that the employment of small wheel-diameters is an accepted practice in India. **This does not necessitate an incline of the cart if the beam in Fig. 1 at p 241 has the correct depth** (B). Ignoring the axle, this should be $H-1/2\,D$ where H is the height of bullocks' necks and D is the wheel-diameter. However, it has been said that in this position the pull may be exerted along the line joining the yoke and the centre of the axle (shown dotted in Fig. 1 at p 241) and therefore the tractive effort may be increased. For a 43 in. bullock and a wheel-diameter of 61 in. normally employed with it in Zone 1, the value of X will be :—

$$X=\sqrt{L^2+B^2} \qquad L=8 \text{ ft } 3 \text{ in.}$$
$$=\sqrt{99^2+13^2} \qquad B=H-1/2\,D$$
$$=43-1/2\times 61$$
$$=\ 99.85 \text{ in.} \qquad =13 \text{ in.}$$

When the diameter is reduced to 50 in. as proposed, the value of X will be :—

$$X=\sqrt{L^2+B^2} \qquad B=H-1/2\,D$$
$$=\sqrt{99^2+18^2} \qquad =43-1/2\times 50 \text{ in.}$$
$$=\ 100.62 \text{ in.} \qquad =18 \text{ in.}$$

$$\therefore \text{ the change is of the order of } \frac{100.62-99.85}{99.85}$$
$$=0.8 \text{ per cent.}$$

The tractive effort of this wheel (61 in. diameter) is 72 lb per ton. Therefore, the increase will be 0.64 lb per ton which is negligible. In practice this will happen *only* if the yoke is sloping, not otherwise.

TRACTIVE EFFORT.

29. The question of tractive effort is more important than that of the inclined line of the pull. This will undoubtedly increase the tractive effort for the same tyre **but if the tyre is widened, the tractive effort of the same wheel will be reduced.** The tractive efforts of these two wheels for 2 in. and 3½ in. tyres, **both employing the same axle equipment,** are as follows :—

61 in. by 2 in. : 72 lb/ton. 61 in. by 3½ in. : 60 lb/ton.

50 in by 2 in. : 81 lb/ton. 50 in. by 3½ in. : 70 lb ton.

Thus **the tractive effort of a 50 in. by 3½ in. wheel will be practically the same as that of a 61 in. by 2 in. wheel in use at present in spite of a reduction of 11 in. in the wheel diameter.** In fact, it will be 2 lb less against an increase of 0.64 lb if a sloping yoke is used as shown above. Therefore, there need be no fear either on the score of an inclined line of the pull or tractive effort so long as the body of a cart is kept parallel to a road surface. This has to be done by inserting over the axle box a wooden batten of a thickness which is half that of the difference between the present and proposed wheel-diameters. This will cost about Rs 5 per cart for which there is sufficient margin in the cost of the proposed wheel.

CARTER'S REACTION

30. **Some people seem to have an apprehension that carters will not agree to a proposal to reduce the present wheel-diameters. Perhaps this fear is based on nothing better than the broad assumption that, since a carter has been used to a particular set-up for years, he will be disinclined to make any change in it. It is, therefore, proposed to discuss the point from all angles :—**

(i) **It is quite true that the carter will not react favourably to any change at first. But this should apply both to the tyre width and the wheel-diameter.** *It* **is not correct to presume that the carter will not object to a wide tyre but will object to a change in the diameter.**

(ii) **It has been shown above that the carters employ certain wheel-diameters at certain stations for the simple reason that those particular diameters suit them in respect of cost and tractive effort for the work offering.**

What the carter is concerned with is the cost and tractive effort of his wheels and not any particular wheel-diameters.

(iii) It has been found that wheel-diameters in use at present vary not only from Zone to Zone but from station to station in the same Zone, the variations being from 7 in. to 23 in. (See Annexure I). **This shows that carters are used to variations in wheel-diameters.**

(iv) The *maximum* variation in the proposed wheel-diameter is only 9 in. (vide para. 25). This is within the range of variations to which carters are used.

(v) The diameter of pneumatic tyred bullock-cart wheels is only about 3 ft and, during the last 20 years or so since pneumatic tyres were introduced, nobody has heard any complaint from the carters on the score of a small wheel-diameter. The reason is that these wheels have roller bearings and, therefore, entail a smaller tractive effort. **So long as a wheel does not entail a greater tractive effort, the carter does not care what the diameter is.**

(vi) During the past six years the Author has had occasion to distribute to carters about two dozen pairs of wheels of various diameters and various tyre-widths. No carter ever asked any questions about the wheel-diameters, but every one wanted to know whether the new wheel was "heavier" than his present wheel.

(vii) It is accepted that, to save damage to road surfaces, a broad tyre is necessary, this has been clearly established in the wear and tear tests. The Author has not come across any wheel-design so far which, for the same wheel-diameter as is in use at present, will provide a wide tyre without exceeding the cost of the present wheels. In fact, it is impossible to do so. Therefore, **a reduced wheel-diameter must be a *sine-qua-non* of broad tyres unless cost is no consideration.**

(viii) The influence of wheel-diameters on the tractive effort of two wheels of the same tyre depends on how the weight of a wheel is distributed If more of it is concentrated at the rim than at the hub, the wheel is easier to move and vice versa. **So long as a proper balance is maintained between the weight of a wheel at the hub and the rim, as in the case of wheels in use at present, one can use any wheel-diameter and any width of tyre provided the cost and tractive effort remain unchanged.**

SUGGESTED WHEEL DESIGN.

31. The following four designs are so far available for comparison with the wheels in use at present :—

(i) The Cumming wheel
(ii) The Raju roller wheel
(iii) The Nageswara Ayyar wheel
(iv) The Morgan wheel

32. Particulars of these have been mentioned in brief at various places in the four previous Papers of this series. It is proposed to discuss them in detail here.

33. **The Cumming Wheel** is an all-steel wheel built for an all-steel-cart. Its tyre consists of two angles welded back to back with mild steel flats for spokes, also welded at one end to the tyre and at the other to a wrought iron hub. This wheel is of no use to the wooden cart in use at present. Also, it makes no provision in its hub for self-alignment and or flexibility. Hence it will not be considered further.

34. **The Raju Wheel** has been described in a previous Paper of this series. The idea underlying the design is to replace a wheel with a roller. The roller tested at Poona was not found to justify any of the claims made on its behalf by the designer.

35. **The tyre of the Nageswara Ayyar Wheel** consists of a channel iron section without a wooden felloe. It is bolted to wooden spokes which, in the wheels sent to Poona for testing, were held in place by wedges nailed to them. (Fig. 1, Plate II). The axle is square and carries two spherical journals on which are mounted two spherical casings (Fig 3, Plate II) bolted to the wedges in between the spokes (Fig. 1, Plate II). A 3/8 in. gap has been provided in the hub and also in the casings to enable the wheel to tilt to produce self-alignment (Fig. 3, Plate II). A 7/8 in. space has been provided on the axle on either side of the wheel for sliding (Fig. 3, Plate II).

The designer makes the following claims:—

(a) By eliminating the wooden felloe, the wheel has been lightened and cheapened.

(b) By providing a gap in the hub and the casings, the wheel has been made self-aligning.

Comments on the Nageswara Ayyar Design.

36. The method of nailing wedges to spokes and passing bolts through the former is unsatisfactory for the following reasons:—

(i) A large number of nails are required to be fixed to ensure a tight fit with spokes. Nails can never be fixed satisfactorily as the angle of fixing has to be different for each nail.

(ii) Nails weaken both spokes and wedges.

(iii) The bolts weaken the wedges. When the hubs were opened at Poona, practically every wedge was found damaged and the wedges carrying bolts were split.

(iv) As a result, most of the spokes were found to be out of alignment.

The only way to ensure stability with wooden spokes is to employ a wooden hub made exactly on the lines of the hub in use at present providing perfect embedment for spokes. No other expedient like wedges is feasible.

37. Tests carried out at Poona have established that the spokes abutting on the bore of the hub of this wheel (Fig. 3, Plate II) touch the square collars of the journal over the axle when the cart is loaded and the corners of the collars interfere with the free movement of the wheel (Fig. 2, Plate II) and increase its tractive effort. Further, they cut into the inner ends of the spokes and wear them. This must be corrected. The designer has accepted this suggestion. In the wheels tested at Poona, the collars extended to about $1\frac{1}{4}$ in. under the spokes (Fig. 3, Plate II). It will be economical to cut this extension and to round off the corners of the axle next to the spokes.

38. When a cart is loaded the axle in this design has been found to descend, as indeed it should do, on account of the gap in the casings. When the wheel moves, the bore in the wooden part of the hub rubs against the steel collars of the journal and is worn by the action of steel on wood. If the collars are cut as suggested above, the axle will do the same. The position is exactly analogous to a wooden wheel without a steel bearing being employed with a steel axle. **To prevent this a steel bearing must be provided in the wooden part of the hub exactly as in the wheel in use at present.** Further, to obtain comparable results, the bearing *must* be of the same thickness, i.e., half inch. The bearing will have to have lugs again as in the present wheel,

Hence, the bore in the wooden hub in this design will have to be $4\frac{3}{4}$ in. as shown below and not $3\frac{3}{4}$ in. as provided in the design (Fig. 2, Plate II):—

Axle	$1\frac{3}{4}$ in. (rounded corners)
Space for free play	1 in. ($2 \times 1/2$ in.)
Bearing	1 in. ($2 \times 1/2$ in.)
Lugs	1 in. ($2 \times 1/2$ in.)
	Total $4\frac{3}{4}$ in.

How to improve it.

39. Employing the same spokes as in use at present, i.e., $1\frac{1}{2}$ in. by $3\frac{1}{2}$ in., instead of 2 in. by $4\frac{1}{4}$ in. employed in the Nageswara Ayyar wheels tested at Poona, the diameter of the wooden hub, required to obtain the same embedment for spokes as in the wheels in use at present, will be as follows:—

$0.785(10.5^2 - 2^2) = 0.785\,(X^2 - 4\frac{3}{4}^2)$ where $10\frac{1}{2}$ in. is the diameter of the hub of the wheel in use at present for a 2 in. bore.

$$X^2 = 129$$
$$X = 11.36$$
say $11\frac{1}{2}$ in.

40. A design of the Nageswara Ayyar type employing a $11\frac{1}{2}$ in. hub, arrived at as above, and providing space for tilting to a maximum camber of 1 in 15 works out as shown in Fig. 2. This figure shows thinner spokes and thinner sections for the casing than used in the wheels tested at Poona and further it gets over all the defects noticed in those wheels and mentioned above, viz.:—

(1) It has a rounded axle.

(2) A wooden hub eliminating wedges and nails.

(3) A bearing for the hub to prevent it from being damaged by the axle.

(4) Bolts above the wooden hub and not passing through it.

(5) A full tilt for a camber of 1 in 15.

(6) No projection of the journals inside the wooden hub.

41. It has been established that a mere tilt will not produce full self-alignment. A slide along the axle is also required.

The designer has accepted this comment and has now provided a 7/8th inch space on the axle for sliding. However, the axle and the collar of the journals are square. This restricts slide and increases the tractive effort during sliding. **To ensure a free and easy slide the axle and the collar of the journals must be rounded, lathe-turned and properly lubricated.**

42. The space on the axle for a slide cannot be left open as it harbours dust. Also, with an open space along the axle, a wheel keeps on banging on the thrust-piece on one side and the pin on the other causing impacts. The slide must be gradual. **For this purpose two rubber paddings in these spaces are essential together with such complimentary fixtures as may be found necessary.**

Lubrication.

43. The design entails lubrication in two places between:—

(i) the axle and the bearing of the wooden hub, } Fig. 2
(ii) the journals and the casing. }

Both these places are extremely inconvenient for proper lubrication, the former because of the half inch gap between the bearing and the axle, and the latter because their surfaces are spherical, **and the lubricant introduced at the apex of the casing is bound to run down to the gap at its lower end.** In spite of the best care it was impossible to retain the lubricant in these wheels in the Poona tests. This is a problem for the designer, whose idea to embed felt in the casing to prevent loss of lubricant does not seem workable for the following reasons:—

(i) the casing is $\frac{3}{8}$ in. thick, hence one cannot employ a felt thicker than $\frac{1}{8}$ in. at the most;

(ii) the felt will get flattened between the casing and the journal as these two surfaces must always remain in contact and, therefore, the lubricant will escape over the felt;

(iii) it will not be easy to renew the felt often without unbolting the casing each time;

(iv) felts are out-moded because this method of lubrication is not efficient.

Bolts and Nuts.

44. The channel rim in this design is bolted to the spokes. In the wheels tested at Poona the bolts were found to have split

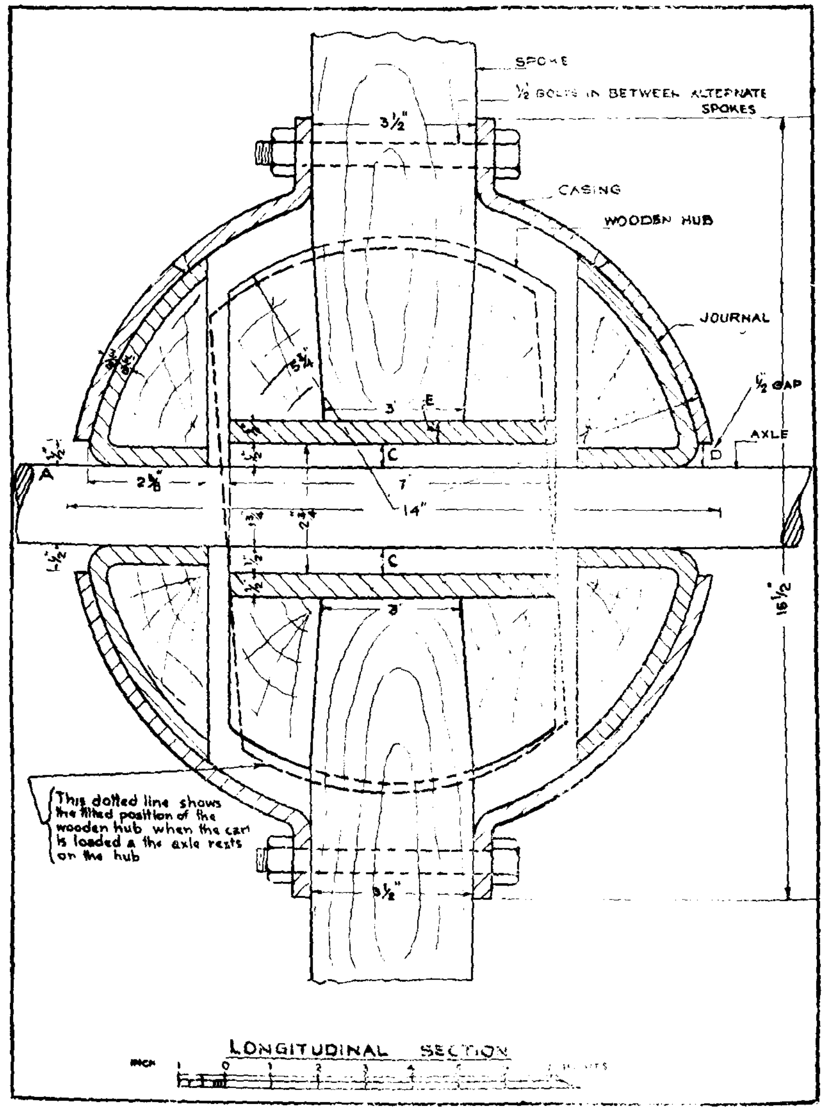


Fig. 2.

the spokes. This is inevitable and will entail frequent renewals of the spokes.

45. The two spherical casings of the wooden hub are also bolted together. In the wheels tested at Poona, the bolts were passed through wedges in between spokes. This objectionable feature can be removed in the way suggested by the Author in Fig. 2. But that fact will not prevent the bolts from getting loose by impact. Tests have shown that if some bolts are even slightly loose, they increase the tractive effort of this wheel. **The expedient of employing bolts is extremely unsafe.**

Wheel Assembly.

46. Assembling the Nageswara Ayyar wheel is a much more cumbersome and difficult process than putting the ordinary present wheel together.

Channel Iron Tyre.

47. The use of a channel iron section in place of a mild steel flat tyre on a wooden felloe is cheaper in initial cost but we have to consider the following counterbalancing factors:—

(i) The sole function of a tyre is to take up wear and tear caused by abrasion. Hence, in respect of renewals, a channel section will behave exactly like a flat tyre. However, **in the case of the latter, the renewal does not extend to the wooden felloe which remains intact while in the case of the channel section it will cover its two legs also.** The quantity of steel in the legs is practically the same as in the tyre. **Hence in the case of a channel iron section the cost of renewal will be double that of a flat tyre.**

(ii) In the wheel in use at present, the tyre is fully supported by the wooden felloe. In the channel iron section of the Nageswara Ayyar wheel, it is unsupported except by the channel legs. It has still to be seen whether such partially supported section of steel of the same thickness will have the same life in respect of wear and tear caused by abrasion as a fully supported flat tyre. If in practice it is found to require a thicker section, then the saving on the wooden felloe will disappear and in addition the cost of renewals will be appreciably **increased.**

(iii) The cost on labour on a normal bullock-cart wheel in Bombay works out as follows:—

	Rs	A	P	Rs	A	P
Fixing tyre	4	0	0			
Fixing hub-bands	6	0	0	10	0	0
Fixing felloe	18	0	0			
Fixing spokes	7	8	0			
Fixing hub	4	8	0	30	0	0
				40	0	0

Thus 60 per cent of the wages of a wheel-wright, excluding the cost of fixing the tyre and hub-bands, is for his work on the felloe. Therefore, the employment of a channel iron section will deprive a wheel-wright of a major part of his work. The Nageswara Ayyar design does not employ hub-bands. Hence, out of Rs 40, which a wheel-wright makes at present per wheel, he will lose in the Nageswara Ayyar design Rs 24 (Rs 18 felloe + Rs 6 hub-bands), i.e., 60 per cent of his wages.

(iv) From the point of view of national economy it is a moot point whether an article made of local wood and providing maximum employment to village carpenters should be replaced with steel which is in short supply and which has a number of uses wherein wood is either uneconomical or unsuitable.

(v) From the point of structural stability, **with wooden spoke one must employ a wooden felloe and a wooden hub to embed both ends of a spoke firmly.** With steel spokes a steel channel or angle-iron felloe is required and a steel hub welded to the spokes as in the Cumming wheel and not bolted as in the Nageswara Ayyar wheel.

(vi) It is a common experience that wheels sink into soft wet surfaces. With a felloe 4 in. to 6 in. deep ordinarily a wheel will not be submerged when it sinks. With a channel section with its legs barely about 2 in. deep, the felloe will not only get submerged but will have to scoop mud when it revolves. This may be a serious matter for the carter.

The Morgan Wheel.

48. This being a patented design, full particulars are not available. It makes use of the wheel in use at present, the only change being in the hub. Here the designer employs a ball and socket arrangement to produce self-alignment.

Comparison of N. Ayyar and Morgan Wheels.

49. The position in regard to the Nageswara Ayyar and Morgan wheels vis-a-vis the wheel in use at present is as follows :—

(i) All the three wheels require a wooden hub, a steel bearing, and a round lathe-turned axle for a slide of the wheel.

(ii) They also require rubber paddings together with such fixtures as each design may entail in the space on the axle provided for a slide.

(iii) The present wheel has no provision for tilting while the other two designs have it. But in both the Nageswara and Morgan wheels, the tilt has been obtained from a gap between the bearing and the axle. Exactly the same thing can be done in the present wheel also.

(iv) The reason why it is not advisable to do so is that it will be impossible to retain the lubricant in this gap. Thus the Nageswara Ayyar and Morgan wheels offer no advantage over the wheels in use at present.

(v) During the past six years the Author had occasions to distribute to carters free of cost about 24 pairs of wheels of various diameters and widths of tyre. In every single case the carter raised the point about the weight of these wheels. The reason is psychological. A broad-rimmed wheel looks more robust and heavier even when it has the same weight as a narrow rimmed wheel of a large diameter. That is why a carter feels it will be injurious to bullocks. If it is our intention to popularise broad-rimmed wheels, we cannot afford to ignore this aspect. For the same diameter and tyre both the Nageswara Ayyar and Morgan wheels are heavier than the present wheel. They are also more expensive, but even if they were not the weight aspect would go against them

HISTORICAL BACKGROUND.

50. The Author came to be associated with the Bullock-cart subcommittee in April 1944 as a result of some suggestions he made at the Gwalior Session of the Indian Roads Congress in October 1943. The position about the Bullock-cart wheel till then was that the Council of the I.R.C. gave annual prizes of pneumatic tyred carts at cattle fairs to encourage carters to employ them in place of the present carts. At the 1941 Session of the I.R.C. at Delhi, Mr. K. G. Mitchell (now Sir Kenneth Mitchell) the then President, referred to this as follows in his address :

"We have to consider how far we can afford to continue to give prizes of pneumatic tyred carts at cattle fairs ; and we have an analogous proposal to consider whether anything can be done to evolve a wheel less damaging to roads than our steel-tyred enemy, that could be made in the village."*

51. The same year Mr. W. L. Murrell read his Paper "The steel tyre problem unfolds" † in which he gave sketches of various ideas of superimposed broad tyres on existing wheels. This was followed by another Paper by him in 1943 on 'Raising the road-rupee ratio with a special reference to the steel tyre", in which he wrote as follows : ‡

"The following are the obviously essential qualifications for any new feature in the bullock cart wheel :—

(i) The feature must result in the minimum possible change in design of the present wheel.

(ii) The new feature must be capable of fabrication by the existing village blacksmiths and wheel-wrights and at reasonable expense.

(iii) The new tyre must be reasonably resistant to wear.

(iv) Repairs must be inexpensive and within the scope of the existing village wheel-wrights.

(v) It would be good if the new tyre could be fitted to existing bullock-cart wheels".

52. Mr. Murrell was then experimenting with broad wooden tyres and exhibited at Gwalior various wheels manufactured by him. Finance for them was provided by the Government of India. Mr. Murrell also suggested in his Paper that there should be a standing bullock cart wheel committee.

53. That was the position when the Author came to be associated with this investigation and in his first Paper in the present series, presented to the Madras Session of the Indian Roads Congress in 1945, he mentioned that the design discussed by him "made use of materials and methods of construction

* (I.R.C. Vol. VII, part 1, P. X, para. 19).

† (I. R. C. Vol. VII, Part I, P. 67).

‡ (I.R.C. Vol. VIII, part 1, P. 48, para. 6).

employed at present by village wheel-wrights in the construction of wheels without in any way disturbing their industry". In all his subsequent Papers he repeated the same fact.

54. The object of presenting this short historical background is to show how the I. R. C. veered round from its initial insistence on pneumatic tyres to a wheel produced in terms of the five requirements enunciated above by Mr. Murrell, which, in fact, were an amplification of what Sir Kenneth Mitchell said in 1941 (para 50). Accordingly the attitude of the Author has been that **we should first and foremost try to improve the existing wheel and not to replace it unless it is found to be beyond improvement to meet our requirements.** The wholesale adoption of pneumatic tyres being out of the question* and the wheel-wright industry having been entrenched in the Indian villages to such an extent that it will be impossible in the interest of village economy either to displace it or to reduce its work, the above obviously is the best approach to this problem under the present circumstances.

PRESENT WHEEL ARRANGEMENT THE BEST.

55. **The Author is convinced that, apart from any other considerations and solely on the grounds of structural stability, performance, and cost, the present bullock cart wheel arrangement is the best of all he has seen and tested so far.** The facts that it has also stood the test of time and the wheel-wright is fully conversant with the technique of making it, are additional grounds in its favour. **In the Author's opinion there is no strong reason to replace the wheel in use at present by one based on entirely different principles.**

56. What is necessary is to provide in it a tilting arrangement without introducing a gap so that the bearing can be efficiently lubricated without loss of lubricant, and will not harbour dust; and the simplest way to do this is to have a tight fitting through cylindrical bearing without a gap. This will facilitate the provision of oil traps to prevent loss of lubricant which would be impossible with a gap. Then, to provide two short rubber sleeves on the bearing, one at each end, to avoid the gap which would otherwise be required to enable the hub to tilt.

FLEXIBILITY.

57. Then, there is the question of flexibility. It is a question of very considerable importance. No wheel of any

* "Our Roads", Modern India Series No. 11, P. 53.

vehicle can at any time be complete without a provision for flexibility in it somewhere. A stiff and rigid assembly like the present bullock-cart is a definite nuisance both to the roads and the animals and it is essential in the interest of both to break this stiffness by introducing an element of flexibility. Flexibility cannot be provided in the chassis for reasons of cost and there is no other place to provide it except the hub. Tests have established that a wheel which has flexibility in its hub behaves materially different from a rigid wheel in respect of tractive effort during travel over pot-holes and uneven surfaces and is easier to pull.

58. Travelling surfaces cannot always be perfectly smooth and are bound to have unevenness, ups and downs, and pot-holes; every bump made on them by a bullock cart wheel, travelling howsoever slowly, is bound to cause damage by impact to the surfaces and injury to the bullocks. Hence, a provision for flexibility becomes essential. One cannot provide springs in a hub and, if they can be provided somehow they will not stay put and will be a nuisance. **The only alternative is rubber, no matter what the wheel design.** Further, the size of the fixture for flexibility should also be the same in every design meant to carry the same load.

SELF-ALIGNMENT & FLEXIBILITY.

59. Hence the proposition reduces to this :

(i) It is our intention to make our wheel self-aligning to ensure a full tyre-contact with the road surface under all conditions of travel, i. e., uneven road surfaces and when the cart is astride the crown of the road.

(ii) Self-alignment requires two movements of the wheel one a tilt in the vertical direction and the other a slide in the horizontal direction. **It is impossible to have self-alignment without these two movements.**

(iii) Each of these movements requires a gap, round the axle for tilt, and along the axle for slide. **It is impossible to have these movements without these gaps.**

(iv) Gaps round and along the axle make lubrication difficult, as they do not permit the lubricant to be retained. They also harbour dust on the axle.

(v) It is possible to close the gap on the axle in the present wheel design by inserting a rubber sleeve in the hub **over** the bearing so that the latter is tight fitting **without a gap** and is thus able to retain the lubricant and exclude dust. The hub resting on the sleeve is free to tilt to the extent required. **This is not possible in the Nageswara Ayyar & Morgan wheels.**

(vi) It is also possible to close the gap along the axle in this design by providing rubber paddings on either side of the wheel over the bearing to prevent its contamination by the lubricant. This is possible in the Nageswara Ayyar and Morgan wheels.

(vii) **Thus in regard to tilting, without loss of lubricant, the present wheel is capable of improvement, but the other two are not. In respect of sliding all the three are on a level provided they employ paddings.**

(viii) It is also our intention to make the wheel **flexible** to absorb impact caused to roads and bullocks' **necks** by travel on uneven road surfaces and pot-holes. **This requires rubber sleeves of the same size over the bearings in all the three designs.**

(ix) Therefore all the three designs require rubber fixtures, both sleeves and paddings, with this difference that the gap in the Nageswara Ayyar and Morgan wheels will still remain open but that in the wheel in use at present will be closed. Therefore, **considered from any angle, it is not possible to do without rubber fixtures so long as one requires self-alignment and flexibility without loss of the lubricant no matter what design one employs, and the design of the wheel in use at present becomes the most economical and most efficient of all.**

RUBBER FIXTURES INEVITABLE.

60. We thus arrive at the conclusion that **rubber fixtures are a sine-qua-non of flexibility and self-alignment if we wish to provide them without loss of the lubricant.** It is necessary to see how they fit in the carter's budget.

APPORTIONMENT OF COST.

61. It has been mentioned earlier that, employing the wheel design in use at present, the cost of the proposed wheels of 3½ in. tyre will be almost the same as that of the present wheels practically at every station. It has also been mentioned that this is exclusive of axle-equipment.

62. **From the carter's** point of view, flexibility and self-alignment can be no concern of his unless to save his bullocks' **necks from impact** [see para. 59 (viii)]. Hence if **the State wish to have flexibility** and self-alignment in the interest of **their roads, the Author** would expect the State to bear the difference in the cost of the proposed and present axle equipments. **If the States are not prepared to do this,** it is difficult to see **how the carter can be expected to bear the cost.** In that event the carter **will have to employ in the proposed wheels** the same axle-equipment as at present.

RENEWALS IN THE PRESENT WHEEL.

63. On the assumption that the State pay this **initial difference,** what will be the carter's liability on maintenance **and renewals of the new wheels?** The proposed wheel is exactly **similar to the present wheel in all respects** except the axle-equipment **hence, the only item to be considered** is the latter. On **this his present liability,** according to information collected from about **100 stations in our survey, is as follows** :—

(i) Renewal of bearings on an average at least **every six months.**

(ii) Lubricant at 25 to 50 lb per annum.

RENEWALS IN PROPOSED WHEELS.

64. The proposed design with rubber fixtures **provides cast iron bearings** (as against mild steel in use at present) **which, in the case of tongas and bullock** cart wheels in Bombay City **have been found to last almost** indefinitely unless there is **a defect in the casting. Hence it** is reasonable to assume that **they will require renewals at such** infrequent intervals that **the cost may be ignored.**

65. **With** a tight-fitting bearing there will be **no gap between the axle and the bearing** and further it will be possible **to provide oil-traps to prevent loss** of the lubricant. It **is surmised that this will save at least half** the quantity of the **lubricant at present used.**

66. **The only item which will** require renewal are **the rubber sleeves and paddings** and not their accessories. **Their cost is practically the same as** that of the present **mild steel bearings. Hence, if the rubber fixtures** last for only about **six months, the cost of renewals in the present** and proposed **wheels will be the same with a saving on the lubricant in hand.**

LIFE OF RUBBER FIXTURES.

67. Will the rubber fixtures last for 6 months? At present and till such time as the design has been fully tried and tested over a period, we can only form an idea on this point from such data as are available. This is as follows:—

(i) A motor tyre does not require re-treading normally under 10,000 to 15,000 miles. This is about 20 months in the case of a cart travelling continuously for 8 hours a day at 3 miles per hour, all the year round with a break for rest of about 20 per cent of the time. The average running of carts even in big cities is much less than this. Hence a 20-month life for rubber seems safe. Those who employ pneumatic tyres on carts say it is from 3 to 5 years.

(ii) The solid rubber tyres employed on tonga wheels are said to last from one to two years.

(iii) The rubber in a pneumatic or solid tyre is subject to continual wear by abrasion on road surfaces. The rubber in the proposed fixtures in the bullock cart hubs will on the other hand not suffer from abrasion at any stage. Its function will be analogous to that of rubber blocks provided on the chassis of a motor car. These are said to last from 5 to 7 years. However, this analogy is not quite applicable since the rubber used on motor cars is hard while that envisaged in the proposed fixtures will have to be softer. Hence, on the analogy of motor cars, the life of these fixtures may be from about 2 to 3 years.

(iv) Unlike the tyres or blocks mentioned above, the proposed sleeves will be protected from sun, rain, dust and lubricant as they will be outside the bearing and also fully encased in the hub.

(v) Similar provision can be made for the paddings also by extending the hub if the cost of doing so is justified by savings on the renewals of paddings.

68. Therefore, *prima facie*, there appears to be a case for expecting a life of from 18 to 24 months for rubber fixtures and, in any case, certainly more than 6 months at which life their adoption becomes justified (See para. 66). This means intensive testing for only six months to see, how the rubber fixtures behave during this period; if they do not behave as expected, why not; and in that case how they can be improved. Everything else in this design, except the rubber fixtures, having behind it the experience of centuries, the field for research is limited to rubber fixtures only. This could be done after the proposed field tests at the Pusa Agricultural Institute are over.

69. In the above discussion the Author has confined himself only to the principles of design and avoided reference to comparative costs of the designs discussed, as he was told on a former occasion that figures of cost at the present stage of this investigation would be only hypothetical.

SUMMARY OF CONCLUSIONS.

70. The Author's findings are summarised as follows:—

(i) There is no need whatsoever to replace the present wheel which is structurally sound, has stood the test of time, can be manufactured in villages with locally available materials, and provides the maximum employment to the village wheel-wright industry.

(ii) Its tyres should be $3\frac{1}{2}$ in. wide with a steel section of $3\frac{1}{2}$ in. $\times \frac{3}{8}$ in. and its diameter should be as shown in para. 25 for various States. This will save about 35 per cent of the damage done to roads in regions where a 2 in. tyre is employed at present.

(iii) This alteration can be carried out by employing the same materials and methods of construction as at present without exceeding, as far as practicable, either the tractive effort or the cost of the present wheel.

(iv) The axle-equipment commonly used at present, which consists of a handmade axle and two mild steel bearings, each about 3 in. long, should be replaced with a lathe-turned axle, a cast-iron through bearing as in tonga wheels and rubber fixtures to provide:—

(1) flexibility to reduce damage to roads and bullocks' necks caused by travel on uneven road surfaces and pot-holes, and,

(2) self-alignment for tilting and sliding to ensure a full tyre contact during all conditions of travel, especially when a cart is astride the crown of a road.

(v) Such a provision does not materially increase either the weight of the hub or the tractive effort of the present wheel.

(vi) The design also ensures efficient lubrication, minimum wastage of the lubricant, and protection from dust blowing on to the axle and bearings. It is capable of employment both on the spoke and arm type wheels and does not entail the replacement of one or the other.

(vii) The cost of the proposed wheel without axle-equipment, which is estimated to be nearly the same as that of the present wheel, should be borne by the carter.

(viii) The cost of the proposed axle-equipment should be borne by the State in view of the saving in damage to road surfaces to the extent of about 35 per cent as mentioned in (ii).

(ix) Maintenance of this axle-equipment is not expected to be more than that on the present wheels taking into consideration the cost of renewing the mild steel bearings in use at present and the quantity of the lubricant used.

(x) Till such time as the State decide to bear the cost of the new axle-equipment and arrangements can be made to manufacture it on the scale required, the proposed wheels should employ the hand-made axles and mild steel bearings in use at present so that the scheme of wide tyres can be brought into effect without loss of time. Any design other than that described above satisfying the same conditions and providing the same performance at a less cost, both initial and recurring, could be adopted.

FINANCIAL IMPLICATION.

71. According to the results of the wear and tear tests † (para. 14, page 84, Annexure II, Part IV of this series), the ratio of the total wear and tear of road surfaces caused by all motor vehicles to that caused by all bullock carts works out to 1 to 10 as shown below :—

(i)	Wear and tear caused by an average All-India cart of 4 ft 6 in. diameter wheels with a 2 in. tyre at 3 M.P.H.	X
(ii)	Wear and tear caused by an average All-India cart of a pneumatic tyred cart for the same gross loading and same speed	0.3 X

† Paper No. 148 printed at pages 65 to 153 of the Journal of the Indian Roads Congress, Vol. XV-Part 1.

(iii)	Wear and tear caused by an average All-India cart of a motor car at 36 M.P.H. (pro-rata to (ii) although the gross loading of a car is less than that of (ii) which was 4000 lb)	3.6 X
(iv)	Wear and tear caused by an average All-India cart of a goods lorry at the same speed [pro-rata to gross loading say 3 : 1 to item (iii)]	10.8 X
(v)	Total wear and tear caused by 1.5 lac motor cars @ 3.6 X per car (iii)	5.40 lac X
(vi)	Total wear and tear caused by 0.2 lac goods lorries @ 10.8 X per lorry (iv)	2.16 „
(vii)	**Total wear and tear caused by all motor vehicles (v + vi)**	**7.56 „**
(viii)	Wear and tear caused by $\frac{2}{3} \times 87$ lac carts of 2 in. tyre (Spoke-type wheels) at X as per (i)	58 „
(ix)	Wear and tear caused by $\frac{1}{3} \times 87$ lac carts of tyres over 2 in. (Arm-type wheels) for an assumed average of 3 in. tyre @ $\frac{2}{3}$ X	19 „
(x)	**Total wear all carts (viii + ix)**	**77 „**
(xi)	**Ratio of (vii) to (x)**	**1 to 10**

72. Against this the following points may be mentioned :—

(i) A large number of carts operate in rural areas having few metalled roads and in many cases no roads at all.

(ii) Many of the rural roads whereever they exist and are used mostly by carts have a poor standard of maintenance.

(iii) Only urban carts ply for 12 months. Most of the rural carts are idle during the monsoon.

(iv) A good proportion of the present maintenance grants is spent on roads carrying mixed traffic.

(v) Before the advent of motor vehicles maintenance grants used to be smaller.

73. It is difficult to make an allowance for these facts but it seems a safe guess to assume that only about 20 per cent of the carts operate throughout the year on roads at present

maintained. In that case the ratio of damage to roads done by all motor vehicles to that caused by the carts operating on the same roads works out to 1 : 2.

74. Total expenditure on roads from different sources in pre-partition India was about Rs 12 crores during the year ending 31st march 1945 (Basic Road Statistics of India, Page 101). According to the I. R. T. D. A., the figure for 1946 was about Rs 14.8 crores. Municipal expenditure on roads during 1941 was 1. 48 crores (Basic Road Statistics of India, P. 102) and may be taken at about Rs 1½ crores in 1946. Thus total expenditure on roads during 1946 was about Rs 16½ crores in pre-partition India excluding what were then known as Indian States. An expenditure of about Rs 3½ crores may be assumed on their account. Therefore, the total road expenditure including maintenance, new construction, etc., from all sources throughout pre-partition India may be taken at about Rs 20 crores per annum.

75. In calculating savings in damage done to roads by vehicles it may be safe to consider only the repair-bill and leave out expenditure on original works and development. This is about 70 per cent of total expenditure, i. e., Rs 14 crores per annum out of Rs 20 crores. On the basis mentioned in para. 73, motor vehicles would account for about Rs 4.66 crores and carts for about Rs 9.34 crores.

76. A 3½ in. tyre produces a saving of about 35 per cent in road repairs over that of a 2 in. tyre. The number of carts employing the latter is about 52 out of 78½ lacs in post-partition India, i. e., about two thirds. The remaining one-third are Arm-type wheels employing wide tyres hence they cannot be taken into account in arriving at an All-India saving. Therefore, an All-India saving by employing the proposed 3½ in. tyre will be of the order of $2/3 \times 35$ = say 24 per cent of Rs 9.34 crores, i. e., about Rs 2¼ crores per annum.

77. The scheme discussed in this Paper envisages that the cost of the proposed wheels excluding the axle-equipment should be borne by the carters as it is estimated to be about the same as at present. That of the axle-equipment providing self-alignment, etc., should be borne by the State out of the savings on roads. The latter work out to about Rs 2¼ crores per annum as shown above. The scheme of improvement will have to be spread over a period corresponding to the life of a wheel so that every time a wheel is due for renewal it receives a subvention. The life of a wheel varies from about 15 to 20 years, say 17 years on an average, and the number of carts is about 87 lacs (we are considering the pre-partition figure as the above figure of Rs 2¼ crores relates to pre-partition period). Therefore, the number of carts to be improved per annum is about 5 lacs against an estimated saving on road repairs of about Rs 2¼ crores. This works out to about Rs 45 per cart or Rs 22 per wheel per annum on the improvement of the axle-equipment.

78. Now there is obviously no point in the State spending this entire saving on improving the axle-equipment since it is the same whether you spend your money on roads and keep the wheels as they are or you save money on roads and spend the saving on your wheels. In fact keeping the wheels as they are being the line of least resistance the State would prefer to do that. The scheme would be economical only if the State can effect the improvement within say half this amount so that the remaining half is the nett saving to the tax-payer. On this reasoning the average margin for improving the axle-equipment works out to only about Rs 10 per equipment both urban and rural, as shown below. The figures are for pre-partition India.

79.	1.	Average road maintenance per annum excluding original works and development: 70 per cent of total of Rs 20 crores (para. 75)	Rs 14 Crores.
	2.	Due to carts assumed at 2/3rd : (para. 75)	Rs 9.34 ,,
	3.	Saving by widening tyres : 24 per cent (para. 76)	Rs 2.24 ,,
	4.	To be spent on axle-equipment : 50 per cent (para. 78)	Rs 1.12 ,,
	5.	Administration charges : 10 per cent	Rs 0.12 ,,
	6.	Balance available	Rs 1.00 ,,
	7.	Number of carts to be improved per annum $(87 \div 17)$ (para. 79)	Rs 5 lacs.
	8.	Amount per cart out of saving on road maintenance	Rs 20
	9.	Amount per wheel out of saving on road maintenance	Rs 10

In the above conclusions, saving obtained from only about 20 per cent of the carts are being utilised for improving all carts and it may be reasonable to press the State to spend the entire savings on improvement if they wish to convert all carts. But in that case the margin cannot be more than about Rs 20 per wheel.

We thus see how small a margin can be expected on the provision of self-alignment and flexibility and why it is essential that the cost of the new wheels must be as near as practicable to that of the wheels in use at present.

80. Now, if we leave out self-alignment and flexibility for the time being, then the State need incur no liability on improvements since the cost of the proposed wheels without a provision for self-alignment and flexibility will be about the same as that of the present wheels and, therefore, can be borne by the carters. But since the wheels are not self-aligning the maximum saving in road maintenance cannot be had. However, there is bound to be some saving on account of the proposed wide tyres since it cannot be said that a cart has necessarily to hug the crown of a road during the entire period of its travel. When it travels on the side of a road its tyres contact a road surface fully without self-alignment. If this period be about half then the saving to the State is exactly the same as with a provision for self alignment. Why should the State go in for it?

81. The only ground on which it can be justified will be flexibility since savings on account of it both to roads and bullocks' necks have not been included in these figures and are extra to them. Thus on financial grounds we arrive at the following conclusions :—

(i) A scheme of wide tyres without a provision for self-alignment in the wheels will produce about the same saving to the State as if self-alignment were provided at the cost of the State.

(ii) Therefore the State can be justified in providing self-alignment only if it provides flexibility also since flexibility will produce more savings.

(iii) In any case a cost on this account of more than Rs 10 to Rs 20 per wheel cannot be justified and then too only if the cost of maintaining the new wheels is not greater than that of maintaining the present wheels.

82. These conclusions suggest that the best course for the time being will be to introduce the scheme of wide tyres intially without self-alignment and flexibility as it is the line of least resistance and then to think of ways and means to introduce self-alignment, etc., gradually when prices are stabilised and the States are better placed in regard to funds. **That is the significance of conclusion (x) in para. 70 above.**

ACKNOWLEDGEMENT.

83 The Author came to be associated with the Bullock-Cart Subcommittee in April 1944 and is submitting this final Paper on this investigation in April 1950. During these six years he has contributed six Papers in all on this investigation, five in the present series and one on road camber which has a bearing on this subject. To assist the Author, the Government of India appointed an Engineer Liaison Officer from 11-12-46 to 31-10-49. He carried out all the tests, except the wear and tear tests, discussed in Parts III and IV of this series and surveyed 31 out of about 100 stations shown on the map in Plate I. Information regarding the other stations was obtained from three sources, the files in the Office of the Consulting Engineer to the Government of India (Roads), the Administrations concerned and from the Author's touring staff. During these six years the Author had discussions with numerous people on numerous points and profited immensely from them. To all of them including the members of the Bullock-Cart Subcommittee, the Test Track staff at Alipore, the Engineer-Liaison Officer and Mr. Taraporevala who designed a Dynamometer for tests, the Author is deeply grateful. He is also most grateful to Mr. McKelvie, who, in spite of his pre-occupations, took great personal interest in this investigation and devoted considerable time to discussions.

Annexure I.

SPOKE-TYPE WHEELS.

Zone I: D = or greater than 2 (H-13 in.)

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference.
		Inches	Inches	Inches	Inches
1.	Trivandrum	40	54	57	+3
2.	Madura	46	66	64	-2 Diff. nominal
3.	Trichinopoly	42	58	66	+8
4.	Mettupalaiyan	46	66	61	-5 Ghat road
5.	Bangalore	45	64	71	+7
6.	Madras	46	66	66	—
7.	Cuddapah	40	54	54	—
8.	Guntakal	41	56	60	+4
9.	Hubli	39	52	56	+4
10.	Goa	39	52	58	+6
	Average:	43	59	61	+2 On average

Zone 2: D less than 2 (H-13 in.) by about 12 in.

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference.
1.	Ratnagiri	36	46	51	+5
2.	Sholapur	44	62	52	-10
3.	Secunderabad	45	64	55	-9
4.	Bezwada	50	74	63	-11
5.	Waltair	51	76	68	-8
6.	Rayaghada	46	66	54	-12
7.	Bastar	42	58	48	-10
8.	Warangal	42	58	54	-4
9.	Adilabad	39	52	48	-4
10.	Aurangabad	39	52	48	-4
11.	Nasik	42	58	45	-13
12.	Bombay	42	58	53	-5
13.	Mahableshwar	48	70	46	-24 Ghat road
	Average:	44	61	53	-8 On average

Zone 3: D less than 2(H-13 in.) by 12 in. to 24 in.

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference.
		Inches	Inches	Inches	Inches
1.	Surat	46	66	48	-18
2.	Yeotmal	45	64	42	-22
3.	Ganjam	54	82	61	-21
4.	Sambalpur	42	58	45	-13
5.	Raipur	45	64	48	-16
6.	Nagpur	40	54	42	-12
7.	Ellichpur	45	64	42	-22
8.	Indore	46	66	54	-12
9.	Itarsi	40	54	39	-15
10.	Jubbulpore	46	66	45	-21
11.	Chirimiri	46	66	45	-21
12.	Satna	40	54	48	-6
13.	Saugor	46	66	45	-21
14.	Ratlam	46	66	54	-12
15.	Udaipur	45	64	42	-22
16.	Salpura	39	52	41	-11
17.	Jhansi	48	70	42	-28
18.	Sawai Madhopur	45	64	40	-24
	Average:	45	63	45	-17 On average

Zone 4: D less than 2 (H-13 in) by 36 in. to 48 in.

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference.
1.	Ahmedabad (A wheel)	57	88	46	-42
2.	Raniwara: 3½ in. tyre:	46	66	32	-34
3.	Jodhpur } S-wheel with broad wooden tyres	52	78	36	-42
4.	Bikaner } S-wheel with broad wooden tyres	55	84	35	-49
	Average:	53	79	37	-42 On average

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference.
		Inches	Inches	Inches	Inches

ARM-TYPE WHEELS

Zone 5: D less than 2(H-13 in.) by about 15 in. to 30 in.

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference
1.	Dehra Dun	48	70	49	-21
2.	Sadulpur	45	64	43	-21
3.	Meerut	54	82	49	-33
4.	Bareilly	51	76	50	-26
5.	Agra	51	76	60	-16
6.	Lucknow	57	88	60	-28
7.	Allahabad (1)	54	82	66	-16
	(2)	54	82	54	-28
	Average :	52	78	54	-24 On average

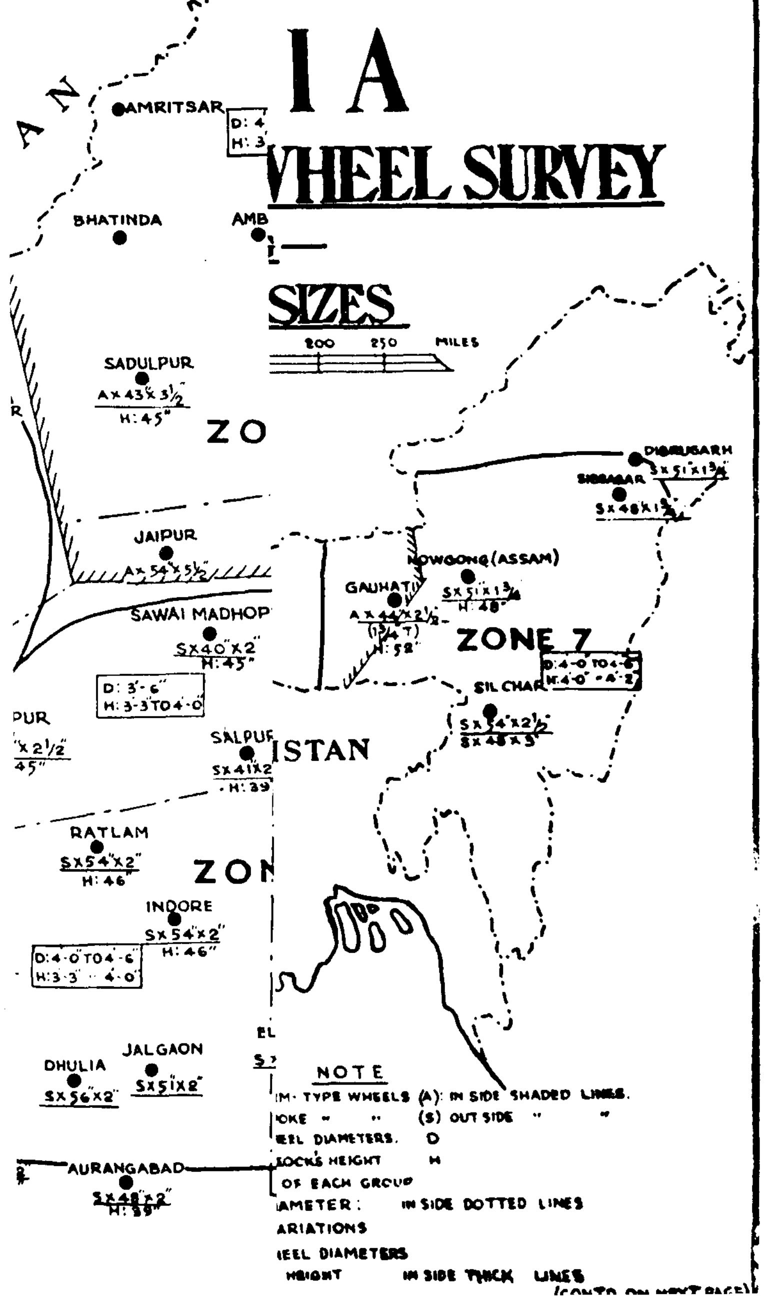
Zone 6: D less than 2(H-13 in.) upto about 15 in.

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference
1.	Fyzabad	45	64	55	- 9
2.	Gorakpur	45	64	58	- 6
3.	Patna	45	64	54	-10
4.	Bakra-Kana	42	58	48	-10
5.	Tatanagar	45	64	48	-16
6.	Bankura	44	62	51	-11
7.	Berampur	46	66	54	-12
8.	Cuttack	48	70	61	- 9
	Average :	45	64	54	-10 On average

Zone 7: D less than 2(H-13 in.) by about 15 in. to 30 in.

No.	Station	Height of Bullocks	D = 2(H-13 in.)	Diameter employed	Difference
1.	Gauhati	52	78	44	-34
2.	Nowgong	48	70	51	-19
	Average :	50	74	48	-26 On average

PLATE I



Paper No. 152.

"Organization of Bridging Activities".

By

G. R. Nangea, M.B.E., M.I.C.E., M.I. Struct. E.

CONTENTS

SYNOPSIS

The Paper attempts to focus attention on the necessity for a continual bridge building programme so that technical skill and "know-how" may develop and be maintained in this country.

2. The Author advocates that loans should be raised for building bridges and paid off by levying tolls on such bridges. Dependence on allotments from State revenues for building bridges will not lead to the fulfilment, in the visible future, of the country's requirements of communications. The Author gives figures based on his experience on toll bridges in the Punjab.

3. Details of the financial and engineering organization and equipment required for continued bridging work are discussed. Equipment suggested to be held in State or Central reserve is listed.

1. INTRODUCTION

1.1. The construction of bridges over streams is an important part of the communication programme of a country. India, with the world's highest mountains, the Himalayas, in the North, the lower ranges of the Aravali and the Vindhyas in the Centre, and the Western Ghats with the plateau of the Deccan in the South, together with its monsoons which distribute rainfall in very varied intensities in different parts of the country, presents indeed, a unique diversity of bridging problems.

advantages are not unaccompanied by correspondingly big communication problems and other complications. To attain parity with other countries in communication facilities numerous streams, big and small, have to be trained, controlled and bridged. India thus needs a very heavy bridging programme before her transport system can adequately link up the different areas of this thickly populated vast sub-continent.

2.5. The numerous factors affecting the stability of structures have each to be studied in detail and unless there are teams of engineers and scientists continuously absorbed in these investigations, the elements may continue to defy control. If, on the other hand, a team of bridge-engineers is employed as a self-contained unit in each State, within a few years they should not only have analysed all the natural phenomena affecting structures in each locality but should also have found ways and means of cheaply overcoming most of the natural impediments in the successful execution of such works.

2.6. Continuous working also leads to standardisation of processes, evolution of new techniques, and collection of skilled specialist technical personnel. The absence of each of the above three factors is chiefly responsible for the high cost of bridge works and the time it takes to execute them. As soon as the bridge engineer in India is assured of continuous work, he will also be enabled to examine his problems well ahead, to reason out the most economical way of solving them, and to start execution at the most propitious time of the year thus enabling quick and economical completion of his undertakings.

2.7. Immediately following the completion of traffic surveys on the different routes, the assessed demand of the traffic for the next 25 years should be placed before the bridge engineer for providing adequate facilities of crossing the streams on the selected route. He will then have to initiate preliminary investigations, planning of the bridge-building programme, and formulation of economical designs for the proposed bridge structures. Continuous observations and correct hydraulic and geological data go a long way towards the exact assessment of the natural forces called into play and this in turn helps the adoption of sound and economical designs.

2.8. With the limited financial resources of our country, it may be a long time before we can afford expensive structures. In some cases the intensity of traffic may only justify temporary or semi-permanent structures with a limited load-carrying capacity. It is this, more even than the demand for permanent structures, which calls for a separate self-contained bridge-building unit in each State, so that a start is made with comparatively cheaper semi-permanent structures where the traffic demand is light to be gradually replaced by permanent structures as the traffic warrants. The components of these released semi-permanent structures can then again be used in some other place. Correct continuous planning and cheap execution of works in the present state of our economy when funds are scarce and the country is not industrially developed are thus a very vital need.

2.9. There has been a great dearth of plant and machinery throughout the country but it must also be admitted that where plant and machinery were procured for a particular work, the maximum benefit could not often be derived from the equipment owing to the discontinuity of the subsequent programme. To keep equipment continually employed and to have it properly looked after is the best way of getting the maximum value out of it. Unless works of a similar nature are expected, any investment in plant and machinery for a single work of short duration is very wasteful.

2.10. Technical personnel of the right type are not very common in India. Men have to be trained as technicians and it is only by the hard way of practical work that mastery in mechanical processes is attained. Unless continuity in work is ensured, the valuable experience of the keen and imaginative worker gets lost to the profession. This is, therefore, an ever-recurring national loss when work is done by fits and starts and needs avoidance particularly when the country has such a huge bridging programme awaiting a start.

3. WAYS AND MEANS

3.1. Bridging is the most expensive single item in the communication programme. Since bridges are essential for opening up the backward areas and ensuring uniform economic development throughout the country, the time has now arrived to examine our immediate requirement in detail and to find ways and means of meeting this demand. By not building a major bridge across a river we force all traffic to go round by a different route which not only takes more time but is also more expensive because of the longer alternative lead. The light industries on which the common man thrives become uneconomical and unworkable and the worker is thrown idle by the higher costs of raw materials and the lack of marketing facilities for his manufactured products. This results in economic backwardness which militates against the general advancement of the country.

3.2. The Nagpur Conference of Chief Engineers in 1943 visualized a total capital expenditure of Rs 45 crores on major bridging which at the present prevailing rates may be taken as Rs 90 crores. Whereas it is accepted that the present economic position of India does not allow of this great expense within the the next fifteen years, it must not be forgotten that there are some schemes which must get through before any economic progress is possible at all. All manufacturing areas must have as straight an access as possible to the main sources of supply and to the centres of consumption for the frutful disposal of their products. For this, the minimum requirement of each State for bridging all the main streams which hamper the easy movement of commodities cannot be less than Rs 3 to 5 crores for the next ten years or so. This would mean an annual expenditure of Rs 30 to 50 lakhs and this sum must be found.

3.3. Can the States meet this demand out of their revenue in addition to their ordinary annual expenditure of Rs 10 lakhs or so on smaller bridges? At least a like sum is also required every year for the new roads leading to these bridges. It is thus clear that to ensure some uniformity in the progress in our communications throughout the country, the States which cannot afford to spend at least Rs 1 crore annually on the new road and bridge development schemes have to seek the help of the Government of India and it is for the latter to find ways and means of supporting them. As a first step, therefore, all States must chalk out their development requirements for the next ten years and, where they fail to finance them fully, the Centre should come to their aid and make their bridging programme a possibility. To foster a judicious spending of the finances of the States on these important links, the Central authority may specify the building of these roads out of State funds simultaneously with the bridges, which are financed from federal funds.

3.4. Are Self-Liquidating Projects Possible?

3.4.1. At most of the places where major streams require immediate bridging, some sort of a ferry service or boat bridge is already being maintained. In all these casses toll is collected. It is strange that, whereas we countenance a toll on a temporary boat bridge or a ferry for an imperfect service across a stream we are shy of collecting tolls on a permanent bridge which, sometimes owing to a lack of finance, remains unbuilt. In a financially prosperous country like U. S. A., even today several main bridges have tolls and most of the biggest bridge schemes had to be launched as self-liquidating projects under a public authority specially constituted by legislation. It is for consideration if the levy of tolls on some of the expensive bridges in this country should not be allowed for 20 to 50 years during which time they pay back the initial outlay rather than postpone building these

bridges and fail to connect the areas so far economically severed or imperfectly linked.

3.4.2. A brief history of the creation of the public authorities constituted for embarking upon two big bridge projects in the U. S. A., and the tolls collected on them is given in Appendix A. Against this, for comparative study, the tolls charged on a ferry and boat bridge at Dera Gopipur on the Jullundur-Hoshiarpur-Dharmsala Road in the Punjab (India) are tabulated below. At this place there is, at present, no permanent bridge across the Beas. The ferry plies during the monsoon, whereas a boat bridge is put up during the dry season.

Rates of Toll

		Rs.	A.	P.
Persons above 3 years age, per head	...	0	0	6
Buffalo, bull or cow ,, ,,	...	0	1	0
Buffalo or bullock (laden) ,, ,,	...	0	2	0
Camel or donkey ,, ,,	...	0	4	0
Camel (laden) ,, ,,	...	0	8	0
Horse or mule (laden) ,, ,,	...	0	4	0
Sheep, goat or pig ,, ,,	...	0	0	3
Motor lorry (empty), each	...	2	8	0
Motor lorry (laden) ,,	...	3	0	0
Motor car (empty), ,,	...	2	0	0
Motor car (laden), ,,	...	2	8	0
Motor cycle (with rider), each	...	0	4	0
Bicycle (with rider), each	...	0	2	0
Other vehicles with 4 wheels each	...	2	8	0
Other vehicles with 2 wheels ,,	...	1	8	0
Head load exceeding 15 seers	...	0	0	6

3.4.3. At some of the boat bridges and ferry services at other sites, the recurring expense borne by the State or Central finances is about three per cent of the cost of a permanent structure. In such cases it should obviously be considered economical and better to raise loans and construct immediately the bridges meeting the interest charges from the same source from which the cost of maintaining the temporary bridge or ferry service is being paid.

3.4.4. Even where the amount of interest is not fully met by the cost of maintenance, as in the case of the Dera Gopipur

Bridge, it may be possible to meet the expense of the initial outlay as suggested below :—

The cost of the proposed bridge at Dera Gopipur is	Rs 18,00,000
3 per cent interest on investment	Rs 54,000
½ per cent repairs and maintenance charges	Rs 9,000
Total yearly liability	Rs 63,000
Total yearly expense now borne by the Provincial Revenues (average)	Rs 38,000
Net amount to be met out of tolls	Rs 25,000 annually
	or Rs 70 per day.

3.4.5. At the toll rates already prevailing on the boat bridge at the same crossing, the project will meet the annual recurring costs if 8 buses, 3 cars, 9 carts and 50 laden donkeys or camels cross it every day. Actually even today, if the crossing were sure and easy throughout the year, there would be more traffic than the bare minimum required for collecting Rs 70 a day. After the bridge has remained in use for some time, many times this traffic is sure to cross it and the expected figure of daily income, even in the first three years, may confidently be taken as :

Lorries	25 Nos. @ 3/-/-	Rs	75
Cars	12 Nos. @ 2/8/-	Rs	30
Carts	50 Nos. @ 1/8/-	Rs	75
Donkeys	200 Nos. @ -/8/-	Rs	100
	Total	Rs	280

3.4.6. The net daily saving available for credit against the capital outlay is Rs 210, which means a yearly income of Rs 76,650. Thus the loan would be paid back in 23 years when the toll could be abolished and also the annual drain on the State revenue, in the form of the maintenance charges of the crossing, would have been reduced from Rs 38,000 to Rs 9,000. The toll will thus amount to self-help which the residents of this area will contribute towards their own future economic betterment and it will, in the long run, also help the State finances.

3.4.7. If this principle is accepted, many self-liquidating projects similar to the billion dollar enterprises of the Americans (vide App. A) should be possible in India in the coming future and the bridge engineer in India would have ensured adequate continuity of work in this important and specialised branch of civil engineering. This may also prove a forerunner to the limited financial self-sufficiency for 'Roads' similar to that enjoyed at present by the Railways and Irrigation Departments. Some day, not in the very distant future, India also must create a Public Authority, in some respect similar to the 'Pennsylvania Turnpike Commission' for administering self-liquidating road projects which would otherwise be delayed indefinitely if she relies on her general revenues alone.

3.4.8. It must again be plainly repeated here that even the U.S.A., the richest country in the world, is unable to finance her big bridging schemes without recourse to loans repayable by tolls. India is comparatively poor and in spite of our best effort she cannot escape being so for at least a decade more. The only two alternatives open to her are :—

(i) Await finances, build few bridges, continue to cross streams by ferries and pay tolls.

(ii) Build bridges and pay tolls.

The progressive New India must choose the latter course. Should, however, her finances improve beyond present expectations, she can give up this irksome necessity any time. In realization of the above a financial statement has been drawn up in Appendix B which may prove of general interest.

4. ORGANIZATIONAL DETAILS OF FUTURE BRIDGE BUILDING ACTIVITY

4·1. Financial Organization

4.1.1. Important hydraulic structures like bridges require detailed investigations and observations of the streams at the prospective sites. The actual construction should be taken in hand in a definite working season when rains and floods are not expected to damage the work while still incomplete and green. Detailed designs have to be carefully worked out and the plant and machinery needed for the successful and economic execution of the difficult works involved in the foundations and superstructures have to be obtained and collected. Unless the bridge engineer is assured of the funds well ahead, all these factors which are beyond his control, not only hamper the work and confront him with difficult problems of execution but also raise the total cost of the structure.

4.1.2. So long as the bridging programme is to be financed from the general revenues, the existing system of grants may continue except that five years' guarantee of funds should be given to the bridge engineer to enable him to plan his works well ahead and ensure prompt and efficient execution during the best working season.

4.1.3. If, however, finances have to be raised by loans on self-liquidating projects, the detailed administration and the management of the whole project including the finances should be entrusted to the engineer-administrator who may be assisted by financial and accounts experts. As the man mainly in charge of the affairs, the engineer also must have an effective say in the deliberations of the public authority vested with powers for raising funds, assessing and collecting incomes, liquidating loans and assuming responsibility for further new projects.

4.1.4. The public authority, when created, will usually have state-wide jurisdiction and to ensure general uniformity on broad lines a central liaison agency may prove invaluable. This agency may also be referred to for seeking independent second advice on new projects for which the authority proposes to assume responsibility.

4.2. Engineering Organization

4.2.1. An efficient unit of bridge engineers would consist of an administrative engineer officer with two to three executive engineers and six to eight assistants, besides the usual skilled technicians for different types of works ordinarily required in bridge construction. Such a unit is recommended for each State.

4.2.2. The maintenance of a proper organization is possible only if at least Rs 30 to 50 lakhs worth of bridgework is annually available. In spite of the general shortage of specialist engineering personnel, once the finances have been assured, it would soon be possible to get together a team of experts, Government officials, private consultants and contractors, who are able to tackle even the very biggest bridge problems that the country may be confronted with. In the course of a few years their experience in varied problems will also enable great progress in the special techniques in numerous bridge building operations.

4.2.3. Besides the engineers, the training of efficient technicians and skilled workmen, who can ensure an efficient and economic execution of difficult foundation works and intricate works in the superstructure is a great problem in itself. Continuous work only can ensure the picking out of useful men and it is the training that they get on actual works that enhances their experience and usefulness Traditions will build up in the course of time but a start must be made as early as possible.

4.2.4. Stress is once again laid on the necessity for continuous work which not only ensures economy in execution but also raises the efficiency of the organization.

4.3. Tools, Plant and Machinery

The work involved in bridge construction may be classed under the following four main sub-heads:—

(i) Site Investigation
(ii) Foundations
(iii) Superstructure
(iv) Approaches

For each of these, varied types of tools, plant and machinery are required. Every year new and more efficient machinery is replacing the old though the basic principles on which the machinery was originally designed usually remain the same. The following notes specify only the basic need in plant and machinery and anything more modern and of improved design should be always welcome.

4.3.1. Site Investigation

The truth of what should be expected at a particular site is found by digging a trial pit and verifying what is hidden underground. Where this becomes too costly, a boring is the next best means of obtaining access to the required point and taking out a sample for examination. The object is to identify and measure the thickness and position of each stratum met with and, therefore, the boring methods adopted should be specially designed for the purpose in view.

4.3.1.1. Ordinary Boring.

For bridge site investigation, a 12 in. diameter bore hole is preferable, although at a great depth, or through rock, a smaller bore only may be economically feasible.

Tubes of the required sizes with flush standard square-thread screwed connections and with shoes and drilling heads are a normal first requirement. Clips for locking a platform on to the tube for loading purposes and tillers and tongs to rotate the tube back and forth are essential accessories.

Power or Hand Rig or Shell and Auger Gear :

This is used in all types of soft ground, sands, and clays, and may also be successfully used to some extent in hard materials. The results are the nearest approach to undisturbed sampling.

Augers can be turned by hand in cohesive materials They are tubular, with opposite cutting edges and with apertures for removing the materials.

Shells are tubes fitted with cutting shoes and "clacks" i.e., hinged flaps close to the shoes. They are used for removing sandy and loose materials and usually require the presence of water. The tool is surged up and down on the end of a rope, a wire, or a rod, and the pumping action forces the materials past the clack.

Sand Pumps are shells fitted with a piston which can be worked up and down and so draw up the sand by suction past the clack.

Chisels: These are of various shapes and designs, heavy or light, and are lifted and dropped to break up hard materials for removal by the shell or auger.

Clay Cutters: Clay can be "cored" by dropping a heavy tube repeatedly.

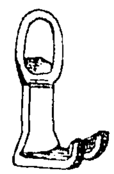
Boring rods: These are necessary for turning augers and at times, for operating the shells or chisels. Those of square section are best. They should be obtained in short lengths capable of being screwed on to give almost any length and able to stand considerable torque. They can be lifted and lowered by means of "lifting dogs". They can be turned by **tillers** or **hand dogs** or by other mechanical means.

Other tools, such as **worm augers** and **shell augers**, have their own place in the boring operations and are needed for different types of work.

Tripods: These must be high enough to allow strings of tools to be hauled up with minimum unscrewing but short enough for easy erection and handling at sites. In most cases one of the three legs usually exists in two parts close together with the winding gear fixed between them. Some of the hand-crabs fixed to the tripods have also fast and loose pulleys for belt-drive.

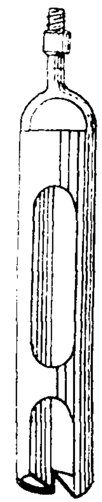
Power Units: These must be portable and yet strong enough for handling different types of bore-holes. Air-cooled units, of 5 to 8 horse power, are useful and 30 cwt piling winches have also proved very valuable where much chisel work is needed.

Clay and sand samplers of standard designs are used for testing samples of clays and sands encountered during boring.

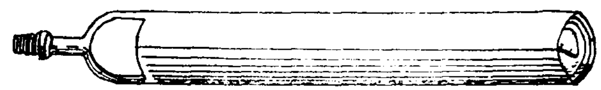


LIFTING DOG

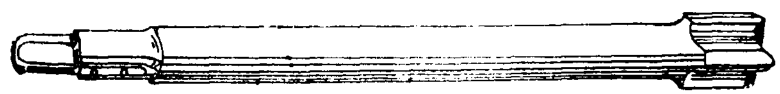
FLAT CHISEL



AUGER



SHELL

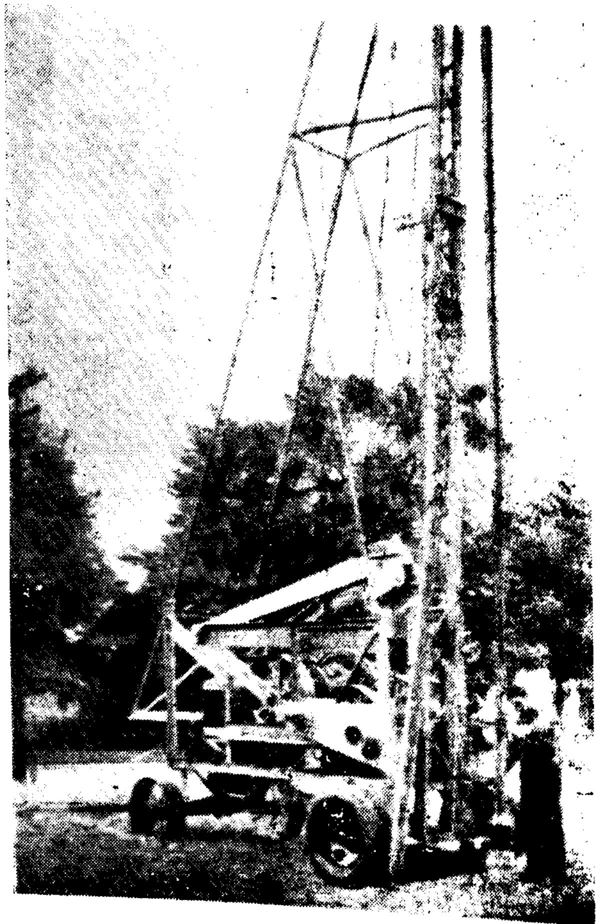


HEAVY CHISEL

4.3.1.2. Percussion Drilling

The rotary motion of a prime mover is turned into an up and down motion by means of a horizontal beam through the end of which the boring rope is passed.

The tools consist of various chisels and bits which are "jumped" up and down, reducing the ground or pebbles and small boulders to a condition in which they can be brought up in a bailer to a shell.



Photograph 1.
Percussion Boring Plant with rig.

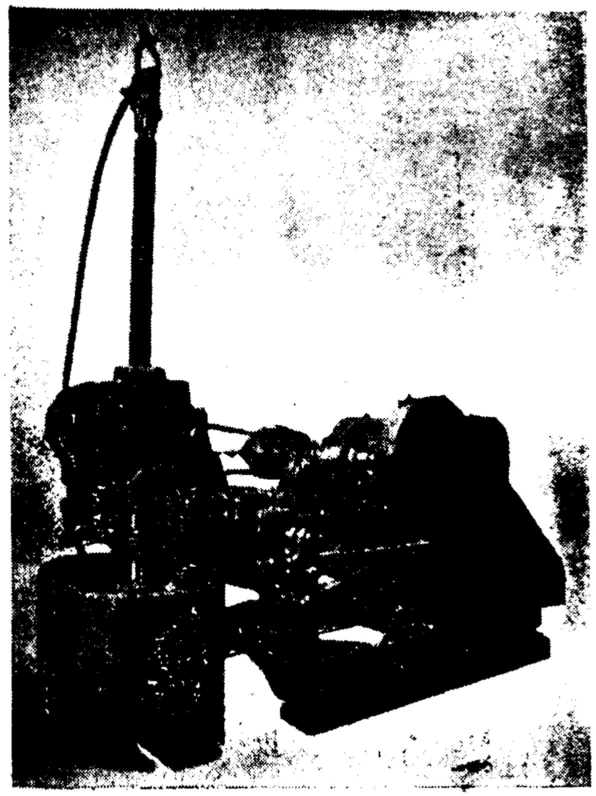
4.3.1.3. Rotary Chilled-Shot Boring

In its simplest form this consists of a turn-table through which passes a square rod which is connected by hollow rods to the tool beneath.

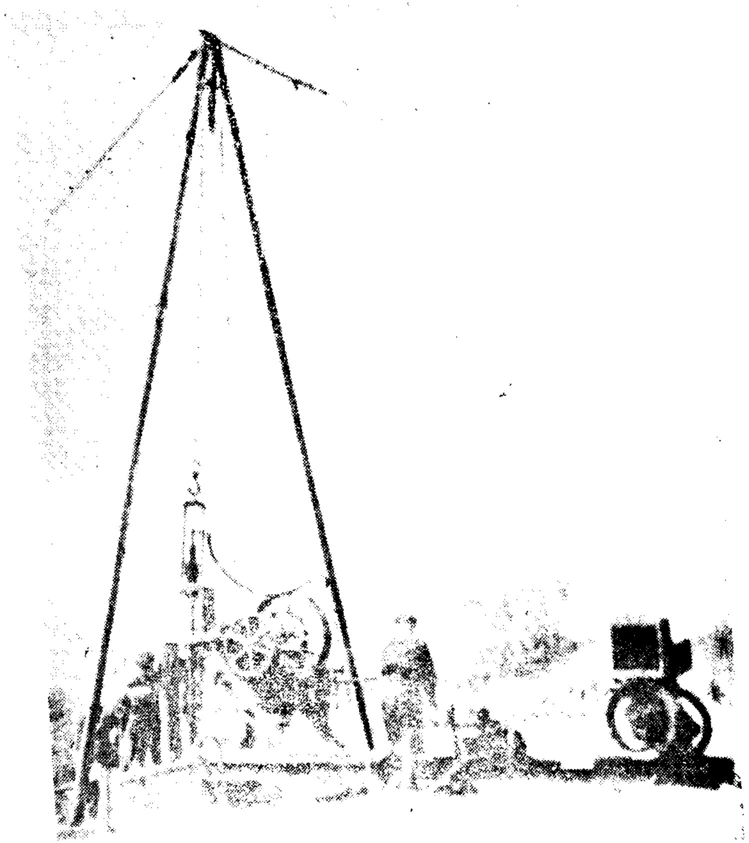
The tool consists of a steel tube with slots at the bottom edge. The chilled shots pass into the slots and then under the edge of the cutter. The rotation causes the shots to wear a slot into the rock and the debris are washed up with the return water.

Thus a core is left which passes up into the tool. Grit is dropped into the circulating water and helps to cause the core to grip the tool. The tool is raised carrying the core with it.

This rig can be adapted to ordinary boring shell and auger work at any time; hence it fits in well with the ordinary boring equipment.



Photograph 2.
Rotary Chilled-shot Calyx Drill.

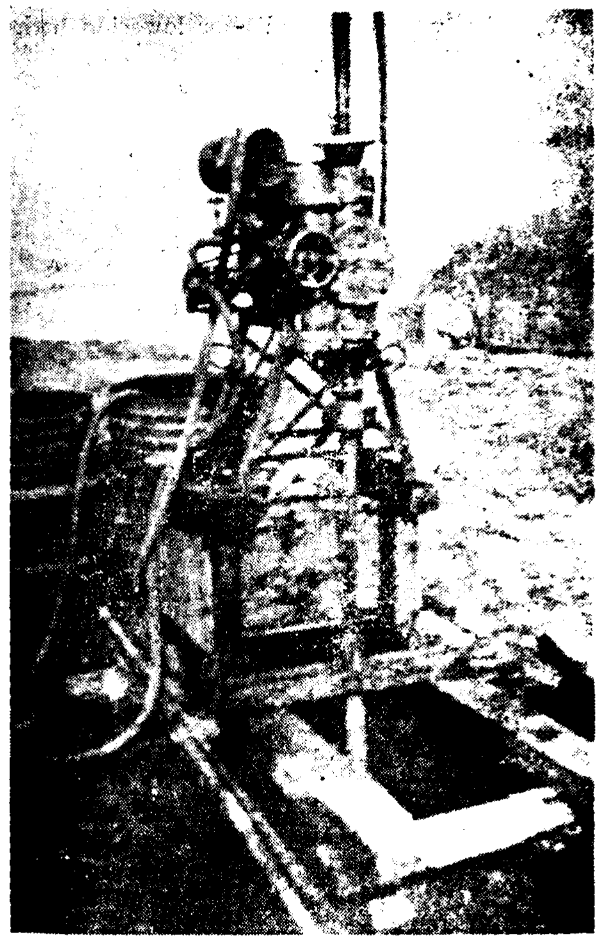


Photograph 3.

Rotary Chilled-shot Drill with Rig and other allied equipment.

4.3.1.4. **Diamond Drilling** by means of rotary bits in which are embedded hard commercial diamonds. As the material is expensive, diamond bits and cores are usually small. The bit is rotated at speeds of up to 1500 revolutions per minute, depending on the kind of rock to be pierced and the size of the bit, whilst a sufficient stream of water under pressure must be pumped down the hollow rods on which the bits are mounted. This is not only to wash the pulverized material to the surface but also to cool the bits, so that if the wash-water is lost in fissures, drilling must stop while the fissure is grouted. A cylindrical core is produced which passes into the core barrel above the bit and is retained there by

the core-lifter, a small spring at the base of the core barrel. This drill is useful in proving and sampling rock in ordinary site-investigation borings to a depth of 100 feet.



Photograph 4.

Light Diamond Drill.

4.3.2. Foundation Work

Any of the following methods may have to be adopted to reach the required depths for laying foundations.

4.3.2.1. Open Foundations

Open foundation work often calls for lowering of the sub-soil water level by pumps and then digging the pit when it is almost dry. It may thus entail the use of heavy pumping equipment. The pumps used should be portable and capable of big discharges so that the least number of pumps is used at each foundation pit. Open foundation work is generally not economical for depths of more than 30-35 feet below the sub-soil water level.

The theoretical suction-head possible for a pump is about 34 feet but even efficient pumps seldom are able to suck more than 25 feet or so. Pumps capable of overcoming 15 to 20 feet suction-head are generally more economical and common. The suction pit (sump) for holding the foot-valve is usually kept two feet below the lowest point of the pit dewatered to enable men to work in an almost dry pit. The foot-valve should be carefully protected by a strainer so that pieces of grit or chips of wood do not get into the valve and stop the pump from drawing water. The pumps used are usually centrifugals driven by diesel or electric power. The modern simplified mechanisms for starting and working the prime movers with self-priming equipment for the pumps should be preferred when obtainable. Since the pumps have to be worked in river beds liable to floods and may sometimes get submerged for even three to four days at a time, those liable to least damage under such conditions should be preferred.

A five-inch suction and four-inch delivery pump is usually the smallest convenient size to use but 8-in. suction and 6-in. delivery pumps are often required where heavy percolation has to be combated.

In one of the pits dewatered in the Author's experience, over 2 lakh gallons of water per hour were being pumped out continuously for 15 days without a break. The pumps obtained for bridgework should be capable of such continuous work with minimum stoppages for oiling and cleaning.

Where power is available, electrically driven pumps are ideal for continuous work. But whenever electric pumps are used, diesel pumps of 25 per cent the total capacity should generally be available as stand-bye or, if there is a breakdown in the power

supply, all the machinery and half-finished work below the sub-soil water level is liable to get sumbmerged. The sides of the pit are then apt to fall in which entails a lot of work before any further progress is possible.

Usually open foundations are adopted in boulder beds where pumping is heavy and some big boulders have to be blasted out. The blast holes are required to be put in quickly for which light drills are required. Petrol rock drills for heavy duty or an electric or a pneumatic drill capable of putting in blast holes one inch in diameter and 3 to 4 feet deep in the toughest rock or stone should be available in such cases.

4.3.2.2. Open Well Sinking

Most of the major bridges in the alluvial plains of India are founded on wells. Generally, the most economical form of well sinking is by open-dredging. Dredgers of different shapes and sizes and of special designs suitable for dredging out different types of strata are available and used. The ordinary Bull's dredger is suitable for work in sand, or sand and clay, or sand and fine grit. Where boulders and bajri have to be dug out, tooth-grabs or grapple-dredgers are needed. Orange-peel grabs or rose shape three-jaw or four-jaw grabs have been known to pick up boulder up to 2 ft diameter.

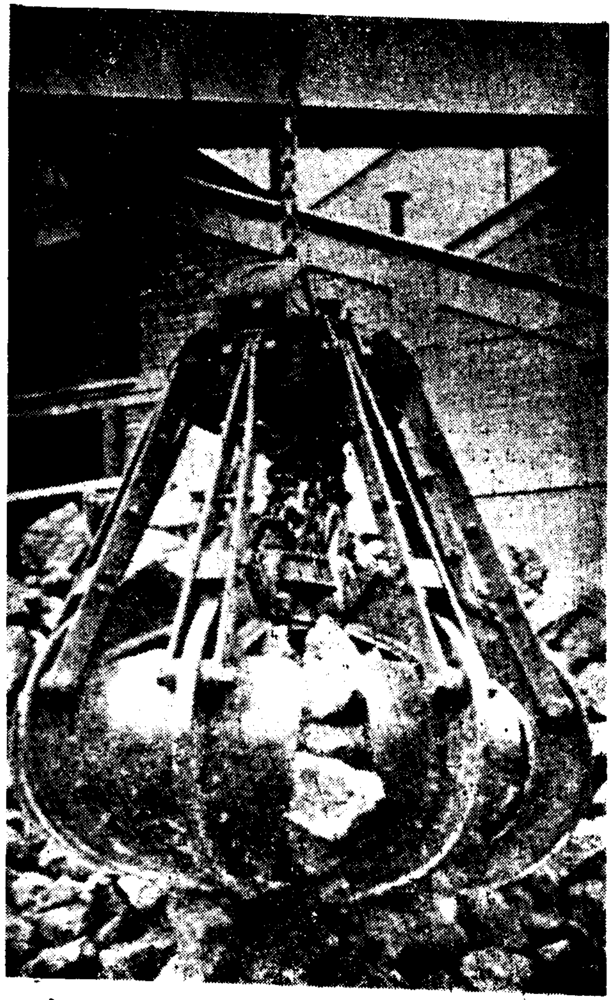
The sizes of the grabs differ with the sizes of the dredge holes in the well. The most common dredge-hole is 8 ft in diameter, whereas 5 or 6 ft diameter circular, or oval, or rectangular holes are also used. Dredgers from 5 to 20 cu. ft (⅕ to 2 tons) capacity are in use according to the requirements of each case.

Sal wood derricks are often used for this type of work, but sometimes steam or diesel cranes are worked with great ease for land or island piers.

Scotch derrick cranes worked by steam hoists on barges are usual for pier work in flowing rivers, but where electric power is available electric hoists fitted on gantries on the sinking set may be preferred.

Ordinarily, 1 to 3 ton double cylinder steam hoists with vertical boilers are economical for jobs in distant out-of-the-way places.

While sinking wells, sometimes some obstruction like an old stump of a tree, or the embedded remains of an old masonry structure, or pieces of rock, etc., are encountered. In such cases divers have to be sent down to clear the cutting edge. These divers work in shifts of four hours or so for depths up to 30 feet below the water level and $2\frac{1}{2}$ hours or less at greater depths.



Photograph 5.

Multi-blade grab operated by ring discharge gear.

Sometimes skin friction makes it very difficult to force a well down. In such cases water jets or compressed air jets worked by

divers and applied all round the cutting edge help the work of sinking.

In boulder beds, the pit in the centre is dredged deep and then a gelignite charge is thrown in the centre of the well to give the well a good jerk. This also helps in moving the boulders from under the cutting edge to the central pit wherefrom they can be dredged out. Rail-cluster-chisels of different designs have also helped dredging and sinking. Sometimes pumping out the well, called "running", is resorted to.

Where the obstruction takes the form of a difficult stratum of unsound rock or huge boulders, it can often only be tackled by completely dewatering the wells. As already stated, for a depth of water up to about 30 feet centrifugal pumps can be usefully employed for this purpose. For greater depths, pulsometer pumps worked with steam boilers may be used. These pulsometer pumps require large volumes of steam and, to achieve good results, big boilers with insulated steam pipes should be procured.

4.3.2.3. Pneumatic Well Sinking

Where open dredging becomes impossible and even dewatering is not practicable and the obstruction is likely to take a long time to remove, pneumatic work may be adopted.

On account of man's inability to work effectively under a pressure exceeding 50 lb per square inch, the maximum depth to which this work can usually be carried out is about 110 feet below water level.

In pneumatic work, the most usual handicap is the small capacity of both muck-lock and man-lock. This becomes most serious with higher pressures which necessitate longer "stage" decompression and shorter working shifts and also when dealing with high shafts which require the passage through the locks of larger quantities of materials and larger numbers of men.

Under Indian conditions, 8 ft diameter dredging holes for the foundation wells are common. A suitable modern air-lock with the necessary shafts and connections may also be adopted for general use.

Good and efficient compressor plant should be provided, with a capacity at least 25 per cent in excess of that required to maintain the working pressure necessary to keep the working chamber reasonably dry, or to provide at least 600 cubic feet of free air per man per hour, whichever may be greater.

With electrical drive there should be a stand-by source of diesel or other power of 50 per cent of the total requirements.

With other than electric drive, not more than 50 per cent of the compressor units should be driven from any one power unit, and where possible, the power units should be made interchangeable.

Man-locks should be of ample capacity, well ventilated, kept in a sanitary condition, and suitably fitted up for the comfort and convenience of the men An electrical or hot-water radiator should be provided for use in the cold weather, or for combating the cold produced temporarily in decompression.

When the working pressure exceeds 25 pounds per square inch, a medical lock should be provided at each working site. The lock should be fitted with all reasonable conveniences and necessities, including a bed not less than six feet in length.

4.3.2.4. Pile Foundations

Pile foundations have often been found more economical than other types and are being more and more often specified for bridges. The piles are provided with suitable shoes and caps designed to the requirements of the strata to be passed through and the weight and design of the monkey or steam hammer.

Pre-cast piles may be placed up-right in wooden frames or on a suitably designed jib which serves as a guide. They are driven by hammers dropped upon them from derricks or cranes. Better results are often obtained when the hammers of the double-acting type are actuated by steam or compressed air. In the double-acting hammer the steam or air raises the striking part and is introduced into the cylinder on the down-stroke as well, supplementing the action of gravity.

In the U. S. A. this type of foundation using steel or concrete piles is very common in situations where deep erosion is not expected.

When using piles cast in situ, Vibro or Franki type or similar types, a bore-hole is put in as described in the boring processes viz., ordinary boring, percussion drilling or rotary

chilled-shot boring and after placing the reinforcement cage the pile is cast in position and the tube is left in place or replaced or withdrawn altogether as the designer of the piles specifies.

4.3.3. Superstructure

The superstructure requires staging and scaffolding for erection. In some steel structures, the individual members are hoisted, laid out, and erected on staging.

Hoists and cranes are necessary for hoisting and placing each member in position and hydraulic or ordinary screw-jacks may be needed for holding them at the exact levels. For riveting, pneumatic riveting hammers of suitable sizes may be used. For drilling holes that may be necessary during erection an electric or pneumatic drill may be useful. In steelwork field welding equipment is usually a necessity. Steel surfaces may also be cleaned by pneumatic brushes.

For concrete structures, the base forms are laid on the supports, and then the reinforcement is laid out, tied, and assembled and, as soon as the side-forms have been erected, the concrete is poured in*

Concrete mixers with concrete conveying equipment and pipes may be used according to the magnitude of the work. Pneumatic concrete vibrators ensure good quality concrete with excellent finish and are recommended for important jobs.

4.3.4. Approaches

So far earthwork in India has mainly been done by manual labour and donkeys. Slowly when this gives place to the employment of mechanical appliances, particularly in the case of high embankments, excavating machinery will come in demand. Some of the modern machines, with 10 yards buckets, can achieve 16,000 cubic yards per day. However, for the present, this machinery may not be considered as essential.

In rocky approaches requiring heavy blasting, quick work can often be ensured if a small portable air compressor with a pneumatic drill is available, capable of putting in one inch diameter, 3 to 4 feet deep bore-holes in rock.

* See also I. R. C. Paper No. 147 entitled "An adaptable steel centring for R. C. Bridges" by A. Nageswara Ayyar—I. R. C. Journal Vol. XIV—3.

Appendix A.

The following is the history of the creation of public authorities constituted for embarking upon two big bridge projects in the U. S. A. :—

1. San Fransisco-Oakland Bay Bridge.

Cost—78,000,000 dollars.

Length of main structure—22,720 feet.

By 1916 the agitation for a bridge had progressed so far that a number of proposals were submitted at a hearing held by the War Department, but no action was taken. At the close of the first World War interest revived. Several promoters sought franchises, the granting of which rested at that time with the Board of Supervisors of San Fransisco County. In 1921, when thirteen applications for franchises had been filed, another hearing before the War Department was held. At this time a ruling was made that no bridge of any kind would be approved north of Hunter's Point. However, the people of the Bay Region would not accept this ruling as final. Proposals were submitted each year, some of which were for a bridge north of Hunter's Point, until by 1928 thirty-eight applications were in the hands of the Board of Supervisors. Meanwhile public interest had turned towards the financing of the bridge by revenue bonds, which had been so successful under the Port of New-York Authority.

In 1929 President Hoover announced the appointment of the Hoover-Young San Fransisco Bay Bridge Commission and the state legislature passed the California Toll Bridge Authority Act, which declared the policy of acquiring all existing California toll bridges, vested the power to grant franchises for private toll bridges in the Department of Public Works created by California Toll Bridge Authority, assigned the Department of Public Works as the agent to design, construct and operate toll bridges, and provided for the financing of contruction by the sale of revenue bonds. The legislature also authorised the California Toll Bridge Authority to "lay out, acquire and construct a highway crossing connecting the City of San Fransisco with the County of Alameda."

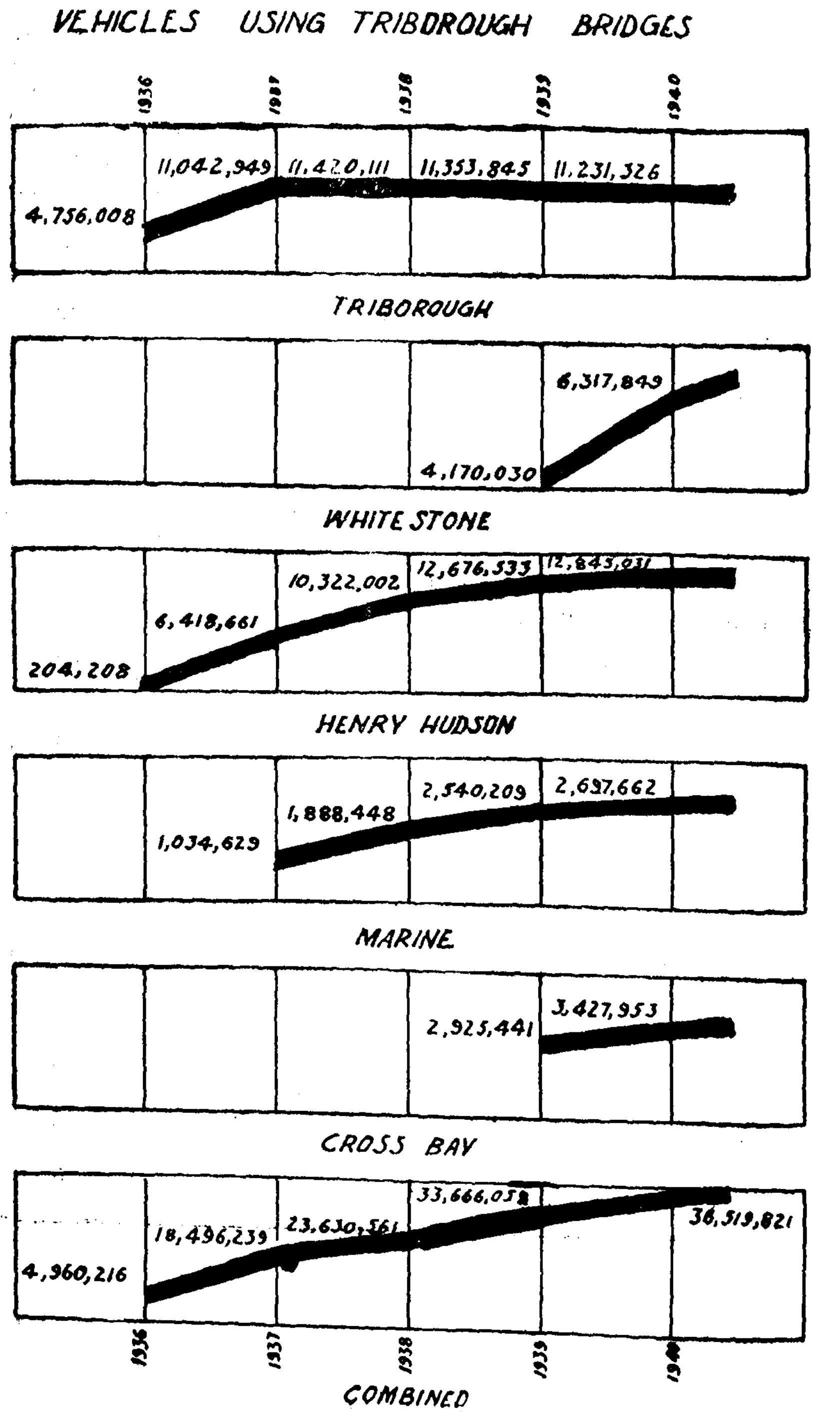
2. Triborough Bridges.

Cost 156,352,000 dollars.

The first definite official action towards constructing the Triborough was taken in the spring of 1927, when general plans were drawn and 150,000 dollars were appropriated by the Board of Estimate and Apportionment for surveys and borings In 1929 the Board ordered the completion of plans and the drafting of a municipal act making the Triborough Bridge a toll project financed by corporate stock or serial bonds, and appropriated three million dollars to begin work. Land for approaches at the bridgeheads in Manhattan and Astoria was then acquired and anchorages were constructed for the main suspension bridge over Hell Gate between Astoria and Ward's Island. Nothing more was done until 1932. In that year the Triborough Bridge was placed at the head of the list of Self-liquidating projects recommended by Governor Lehman's State Emergency Public Works Commission to the Reconstruction. As the city was unable to finance the undertaking with its own bonds or corporate stock, it was agreed that an Authority would be established by state law having the power to issue bonds secured by tolls and which would not involve the credit of the city. The city was, however, authorized to contribute toward the cost of land. The Reconstruction Finance Corporation, which had been studying the project, recommended it to the Public Works Administration which in turn agreed to finance the bridge by means of a 35,000,000 dollar loan and a federal grant equal to thirty per cent of the amount actually expended for labour and materials.

The table below and graphs on page 310 give some very interesting information about the Triborough Bridge Authority bridges in America.

Total Rates	Triborough and Bronx. White-stone Bridge	Herry Hudson Bridge	Marine Part and Cross Bay Bridge
	cents	cents	cents
Passenger automobiles, all types taxi cabs, ambulances, hearses and horse-drawn vehicles.	25	10	10
2-axle trucks, registered weight 2 tons and under.	25	Only pleasure cars and motor cycles allowed	10
Trucks, registered weight over 2 tons to and including 5 tons.	35		25
Trucks registered weight over 5 tons.	50		35
Chartered buses.	50		35
3-axle trucks, tractors or passenger automobiles with semi-trailer.	60		40
4-axle trucks, tractors or passenger automobiles with trailers.	75		50
Franchise buses.	25		10
Motor cycles.	15	10	10
Bicycles.	10	—	5



FINANCES OF THE TRIBOROUGH BRIDGE AUTHORITY

(i) Where the money came from :

		dollars
The Authority sold \$ 98,500,000 of Triborough Bridge Authority bonds to a syndicate of bankers on February 21, 1940 to redeem \$ 53,000,000 of old Triborough Bridge Authority bonds and \$ 18,000,000 of old New York City Parkway Authority bonds and provide additional money for new construction, the proceeds of the sale being		98,586,000
From the Federal Administration of Public Works a grant of 30 per cent on amounts actually expended for labour and materials in constructing the Triborough Bridge ...		9,200,000
Before the Triborough Bridge Authority was created in 1933, the City of New York appropriated for the Triborough Bridge ...		5,400,000
After the Authority was created, the City appropriated for land :		
For the Triborough Bridge	\$ 10,700,000	
For the Cross Bay Parkway Bridge and Rockaway Improvement	3,100,000	
		13,800,000
Revenues—July 11, 1936 to April 1, 1942. (Estimated by the Refinancing Agreement of February 13, 1940. The Authority was authorized to expend all or any part of the existing balances of Revenues and of the Revenues from them to April 1, 1942, for additional construction work or any other lawful purpose) ...		29,366,000
Total money available to the Authority :		156,352,000

(ii) How this money was spent :

Construction of the Triborough Bridge:

Real Estate	23,700,000	
Construction including engineering and overhead	36,600,000	
		60,300,000

		dollars
		B/F 60,300,000
Construction of the Henry Hudson Bridge:		
Real Estate	635,000	
Construction including engineering and overhead	4,465,000	
		5,100,000
Construction of the Marine Parkway Bridge and Improvement at Jacob Riis Parking Field :		
Construction including engineering and overhead		6,000,000
Construction of the Bronx-Whitestone Bridge:		
Real Estate	1,070,000	
Construction including engineering and overhead	16,715,000	
		17,785,000
Construction of the Cross Bay Parkway Bridge and Rockway Improvement:		
Real Estate	4,600,000	
Construction including engineering and overhead	5,600,000	
		10,200,000
Cost of redemption of old bonds and successive refinancing of $ 35,000,000, 53,000,000 and 71,000,000		11,340,000
Construction of Brooklyn Elevated Parkway		15,000,000
Construction of Emmons Avenue Improvement		2,955,000
Construction of Hutchinson River Parkway Extension		7,148,000
Construction of Saw Mill River Parkway **Improvement**		283,000
Construction of Improvements to other connections and the Authority's facilities		2,015,000
Interest on bonds paid and to be paid from revenues to April 1, 1942.		14,001,000
Operating Expenses (July 11, 1936 to April 1, 1942).		4,225,000
	Total	$ 156,352,000

(iii) How the $ 98,500,000 will be repaid :	dollars
The **$ 50,000,000 3½ per cent Sinking Fund bonds** are due February 1, 1980, Semi-annual payments into the sinking fund beginning August 1, 1944, at ...	80,000
And rising to a maximum on February 1, 1980, of	2,330,000
Interest at 3½ per cent to be paid semi-annually on the unredeemed balance will amount in the first year to ...	1,625,000
And in the last year to ...	6,300
The **$ 48,500,000 of Serial Bonds** will be repaid : $ 8,500,000 at 2¾ per cent will be amortized over eight years by payments beginning February 1, 1945, at ...	800,000
And rising to a maximum on February 1, 1952, of ...	1,275,000
$ 40,000,000 at 3 per cent will be amortized over twenty-three years by payments beginning February 1, 1953, at	1,325,000
And rising to a maximum on February 1, 1975, of	1,925,000
Interest at 2¾ per cent on the $ 8,500,000 issue and 3 per cent on the $ 40,000,000 issue to be paid semi-annually on the unamortized balance will amount in the first year to ...	1,433,750
And in the last year to	4,812

Appendix B.

As already stated in paragraph 3.4.5 of the Paper, a daily income of Rs 280 may be expected from tolls on the Dera Gopipur Bridge on the Jullundur-Hoshiarpur-Dharamsala road. The cost of the Bridge will be about Rs 18,00,000.

This means a yearly income of Rs 1,02,200 which is 5·7 per cent of the investment. Similar results could be expected on many other similarly situated river crossings.

It is therefore recommended that India launches on a sustained programme of bridge building at a rate of Rs 3 crores per year and finances it by loans repayable by leaving tolls on all the new bridge structures costing more than Rs 2 lakhs each. The tolls will continue for a maximum period of 50 years on each bridge or group of bridges when they must be lifted.

Attached is a financial statement (on page 315) showing the amounts of loans required per year and the extent to which total loans are necessary and how the bridge building activity could be perpetuated after gradually raising nearly 60 crores in 33 years. In working out this statement 3 per cent compound interest and 0·5 per cent miscellaneous and maintenance charges of bridges have been allowed. The loans will be raised at a rate of Rs 3 crores per year which will gradually be reduced as shown in the statement till Rs 57,59,36,000 in all have been raised in 33 years. The repayment will start in the thirty-fourth year and the total loan would have been completely paid back by the fifty-seventh year.

As will be seen in the statement, when the toll is lifted on 50 year old bridges, the income from toll in the fifty-first year will get stabilized at Rs 8,40,00,000 (as Rs 3 crores worth of toll bridges would have been added and Rs 3 crores worth of bridges would have been exempted from the toll).

In and after the fifty-seventh year, there will be a perpetual yearly income of Rs 8 crores and Rs 40 lakhs available for continued bridge building activity. By that time the country would also have 171 crores worth of bridges which will definitely place India in a proud position relative even to the most advanced countries of the world. At that stage, if India chooses, she can retain the toll for the major industrial and agricultural traffic only or she can abolish it altogether. If the bridge building activity is continued at the same rate of Rs 3 crores per year, Rs 5,40,00,000 will be available for other communication activities.

12	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
13	39,00,00	36,00,00	2,01,60	35,67,41	1,07,02	36,00,00	18,00	34,90,83	
14	42,00,00	39,00 00	2,18,40	37,90,83	1,13,72	39,00,00	19,50	37,05,65	
15	45,00,00	42,00,00	2,35,20	40,05.65	1,20,17	42,00,00	21,00	39,11,62	
16	48,00,00	45,00,00	2,52,00	42,11,62	1,26,35	45,00,00	22,50	41,08,47	
17	51,00,00	48,00,00	2,68,80	44,08,47	1,32,25	48,00,00	24,00	42,95,92	
18	54,00,00	51,00,00	2,85,60	45,95,92	1,37,88	51,00,00	25,50	44,73,70	
19	57,00,00	54,00,00	3,02,40	47,73,70	1,43,21	54,00,00	27,00	46,41,51	
20	60,00,00	57,00,00	3,19,20	49,41,51	1,48,25	57,00,00	28,50	47,99,06	
21	63,00,00	60,00,00	3,36,00	50,99,06	1,52,97	60,00,00	30,00	49,46,03	
22	66,00,00	63,00,00	3,52,80	52,46,03	1,57,38	63,00,00	31,50	50,82,11	
23	69,00,00	66,00,00	3,69,60	53,82,11	1,61,46	66,00,00	33,00	52,06,97	
24	72,00,00	69,00,00	3,86,40	55,06,97	1,65,21	69,00,00	34,50	53,20,28	
25	75,00,00	72,00,00	4,03,20	56,20,28	1,68,61	72,00,00	36,00	54,21,69	
26	78,00,00	75,00,00	4,20,00	57,21,69	1,71,65	75,00,00	37,50	55,10,84	
27	81,00,00	78,00,00	4,36,80	58,10,84	1,74,33	78,00,00	39,00	55,87,37	
28	84,00,00	81,00,00	4,53,60	58,87,37	1,76,62	81,00,00	40,50	56,50,89	
29	87,00,00	84,00,00	4,70,40	59,50,89	1,78,53	84,00,00	42,00	57,01,02	
30	90,00,00	87,00,00	4,87,20	60,01,02	1,80,03	87,00,00	43,50	57,37,35	
31	93,00,00	90,00,00	5,04,00	60,37,35	1,81,12	90,00,00	[illegible]	[illegible]	
52	1,56,00,00	1,50,00,00	8,40,00	[illegible]	[illegible]	1,56,00,00	78,00	14,43,35	
53	1,59,00,00	1,50,00,00	8,40,00	18,49,85	55.50	1,59,00,00	79,50	10,26,15	
54	1,62,00,00	1,50,00,00	8,40,00	14,43,35	43.30	1,62,00,00	81,00	5,97,93	
55	1,65,00,00	1,50,00,00	8,40,00	10,26,15	30.78	1,65,00,00	82,50	1,58,37	
56	1,68,00,00	1,50,00,00	8,40,00	5,97,93	17,94	1,68,00,00	84,00	—	All loans liquidated
57	1,71,00,00	1,50,00,00	8,40,00	1,58,37	4.75				

All loans paid back and stable income available for further communication activities. **Assests: Rs 2,91,38,000.**

NOTE 1:—From the 51st year when the toll is abolished on 50-year old bridges, the income will get stabilized @ Rs 8,40,00,000 per year and, as shown above, all loans would have been paid back by the 57th year. At that time the policy of levying toll will require careful reviewing.

NOTE 2:—Above figures are correct up to the nearest thousand.

Note.—All the above is based on the calculations of income on the traffic expected within the first three years of the building of Dern Gopipur bridge at the rates of toll actually charged on the present ferry and boat bridge there. The traffic may further gradually increase manifold as the communications improve and the area advanced economically. Before launching on a comprehensive programme of this nature, the rates of toll and the mode of realization of the same will need careful examination. Perhaps some bridges on a route may have to be grouped together so that not more than one toll is charged in a journey of less than 50 miles.

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(14) T.J. Evans	FINANCING PENNSYLVANIA'S SUPER HIGHWAY
(15) Pennsylvania Turnpike Commission	GENERAL INFORMATION

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(ii) Ingersoll-Rand Co., Ltd., New York.

(iii) Joseph West-wind & Co., Ltd., London.

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RESEARCH PUBLICATIONS

USE OF STABILIZED SOIL IN ENGINEERING CONSTRUCTION

Section IV—Shrinkage of Compacted Soils.

BY S. R. MEHRA & H. L. UPPAL,
SOIL RESEARCH LABORATORY, KARNAL.

A freshly compacted soil mass decreases in volume, as it is allowed to dry. This reduction in volume or shrinkage varies from soil to soil.

When soil is required to be used as a construction material, and especially in stabilized soil walls, it is very essential that shrinkage on drying should be as little as possible, as shrinkage is accompanied by formation of cracks.

Experiments were, therefore, conducted to get an idea of shrinkage of various soils, when compacted at different moisture contents including the optimum and a small range on either side of optimum with a given mode of compaction.

The types of soils selected for the study are described in Table 1.

Table 1.

Showing analysis of soils selected for the study

Serial No.	Description	Liquid limit	Plasticity index	Sand content	Optimum moisture	pH	Exchangeable Ca, m.e/100 gms	Exchangeable Na+k m.e/100 gms	Total soluble solids
1	Sandy loam	21.8	4.9	65.0	13.0	8.65	4.2	0.90	Negligible
2	Silty loam	27.5	8.7	18.8	12.0	8.40	7.0	1.60	,,
3	Silty clay loam	37.0	17.3	8.4	12.0	8.85	14.8	1.10	,,
4	Loam	25.7	9.5	43.39	12.0	8.10	8.7	1.14	,,

Blocks of all the four types of soils mentioned above, were made by taking 200 gms of dry soil, and compacting at increasing percentage of moisture. The compaction was with 40 blows of a 5.5 lb weight dropping through a height of 15 inches.

The volume of the blocks was determined immediately after compaction by the mercury cup method as suggested by 'Standards for Testing Soils,' and also after drying them in the oven to constant weight at 105—107 deg. C. The results are given in Table 2 and reproduced in Fig. 1.

Table 2.

Showing shrinkage of soils at different moisture of compaction.

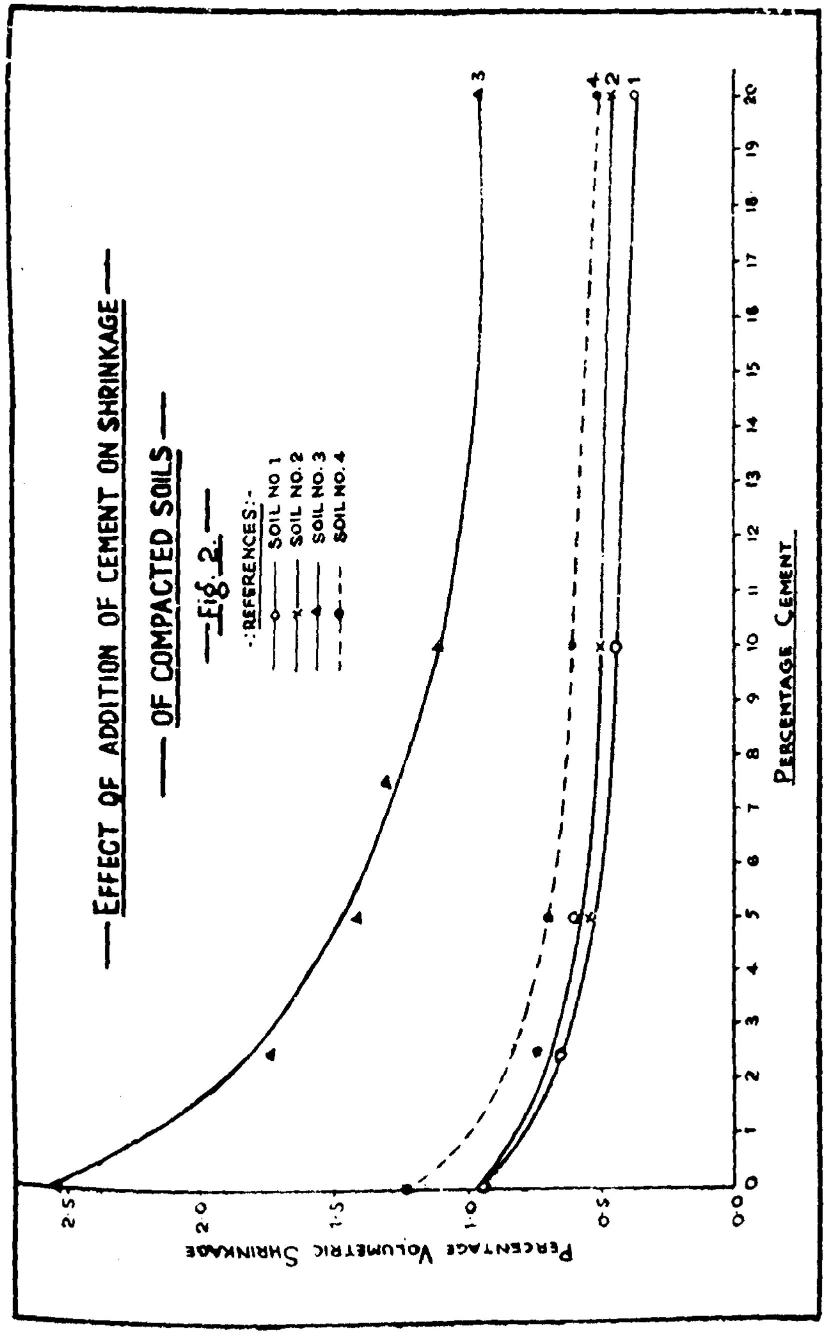
Soil No.	Per cent moisture of compaction	Density at the time of compaction	Volumetric shrinkage
1. Sandy loam	7	1.78	0.16
	9	1.82	0.41
	11	1.87	0.62
	13	1.90	0.94
	15	1.89	1.84
	17	1.83	2.72
2. Silty loam	6	1.61	0.15
	8	1.64	0.30
	10	1.67	0.50
	12	1.69	0.90
	14	1.69	1.17
	16	1.67	1.36
3. Silty clay loam	8	1.72	0.91
	10	1.75	1.47
	12	1.82	2.50
	14	1.81	3.11
	16	1.79	6.07
4. Loam	6	1.79	0.20
	8	1.85	0.59
	10	1.95	0.80
	12	1.96	0.15
	14	1.90	2.25
	16	1.84	3.69

Note :—The figures underlined denote optimum moisture content.

Table 3.

Showing effect of increasing percentage of cement on the shrinkage of compacted soil.

Soil	Per cent Optimum moisture	Percentage cement used	Density	Volumetric shrinkage percentage
1. Sandy loam	13.0	0	1.91	0 96
	13.0	2.5	1.88	0.66
	14.0	5.0	1.88	0.61
	14.0	7.5	1.88	0.50
	14.0	10.0	1.87	0.46
	14.0	20.0	1.87	0.37
2. Silty loam	12.0	0	1.71	0.95
	12.0	2.5	1.70	0.66
	13.0	5.0	1.69	0.55
	13.0	7.5	1.69	0.51
	13.0	10.0	1.69	0.51
	13.0	20.0	1.66	0.47
3. Silty clay loam	12.0	0	1.84	2.54
	12.0	2.5	1.82	1.74
	12.0	5.0	1.81	1.42
	13.0	7.5	1.78	1.31
	13.0	10.0	1.76	1.11
	14.0	20.0	1.74	0.96
4. Loam	12.0	0	1.95	1.23
	12.0	2.5	1.91	0.75
	13.0	5.0	1.89	0.69
	13.0	7.5	1.88	0.66
	13.0	10.0	1.88	0.62
	13.0	20.0	1.87	0.51



It is also seen that the further addition of cement does not reduce shrinkage to any marked degree and, therefore, it will not be normally found economical to increase the quantity of cement beyond 2.5 per cent for the single purpose of reduction in shrinkage. The effect of wetting and drying was also tested with all the four soils with increasing percentage of cement and it was noticed that 2.5 per cent cement was just enough, to keep within safe limits the expansion of the cement soil blocks, when fully saturated with water, except in the case of silty clay loam soil (No. 3) which required a cement concentration of a little more than 5 per cent. This point is to be taken into consideration in areas of very heavy rainfall.

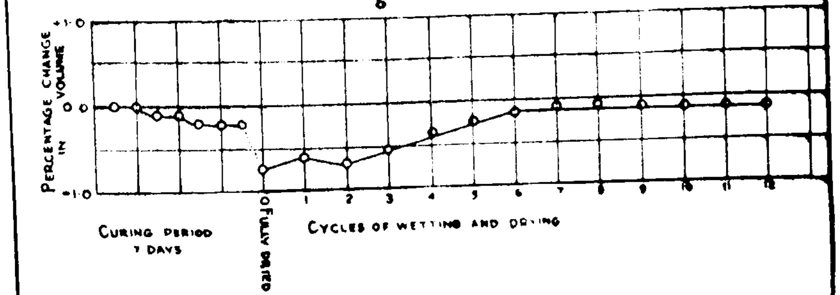
Of all the four soils used in the study, it is the loam soil (No. 4) which appeared most suitable for construction work on account of its proper gradation of particles and because of its containing a sufficient quantity of clay. Since 2.5 per cent cement has been shown to be necessary to provide resistance to water action, the detailed study of shrinkage during curing and subsequent wetting and drying has been confined to loam soils mixed with 2.5 per cent cement only. The results obtained for shrinkage during curing and through cycles of wetting and drying are given in Table 4 and reproduced in Fig. 3. The changes in volume are relative to the original volume at the time of compaction.

Table 4.

Showing shrinkage of loam soil at various stages.

Soil	Shrinkage during curing period		Shrinkage during wetting and drying	
	Period	Per cent	No. of cycles	Per cent
Loam Soil	1st day	Nil	1	0.58
	2nd day	Nil	2	0.68
	3rd day	0.08	3	0.56
	4th day	0.08	4	0.37
	5th day	0.18	5	0.24
	6th day	0.18	6	0.12
	7th day	0.18	7	0.09
	Total ...	0.70	8	0.10
			9	0.10
			10	0.13
			11	0.14
			12	0.14

It will appear from Table 4 that there was a definite, though small, shrinkage during the curing process and, when exposed to repeated cycles of wetting and drying, there was a further shrinkage in the first few cycles and then there was a slight tendency towards expansion. The resultant shrinkage will be such that in a



—GRAPH OF PERCENTAGE CHANGE IN DRY VOLUME—
—AT THE END OF—
—EACH CYCLE OF WETTING AND DRYING—
Fig. 3

long wall of a house, for instance, haphazard cracking will take place as the mass dries up. Such cracking is liable to produce conditions of instability and it is therefore necessary to control this haphazard cracking and localize the cracks at suitably designed joints provided at predetermined intervals. From actual construction experience it has been found that the maximum length of the walling which can dry out without cracking is about 6 ft in case of loam soil (No. 4) and, therefore, it is recommended that vertical joints should be provided at intervals not exceeding 6 ft.

ELIMINATION OF SHRINKAGE

With the object of drastically reducing shrinkage by chemical means to provide joints at longer distances, the addition of plaster of Paris was tried during compaction. Plaster of Paris expands on setting and, therefore, should compensate for the shrinkage if a predetermined amount is added during compaction.

Since at least 2.5 per cent cement is essential to create resistance to water, the effect of plaster of Paris was tried during compaction of soils containing 2.5 per cent cement. The results obtained with all the four types of soils with increasing percentage of plaster of Paris are given in Table 5 and reproduced in Fig 4. It may be pointed out that a silty loam slightly different from the one used previously was used in these tests.

Table 5.

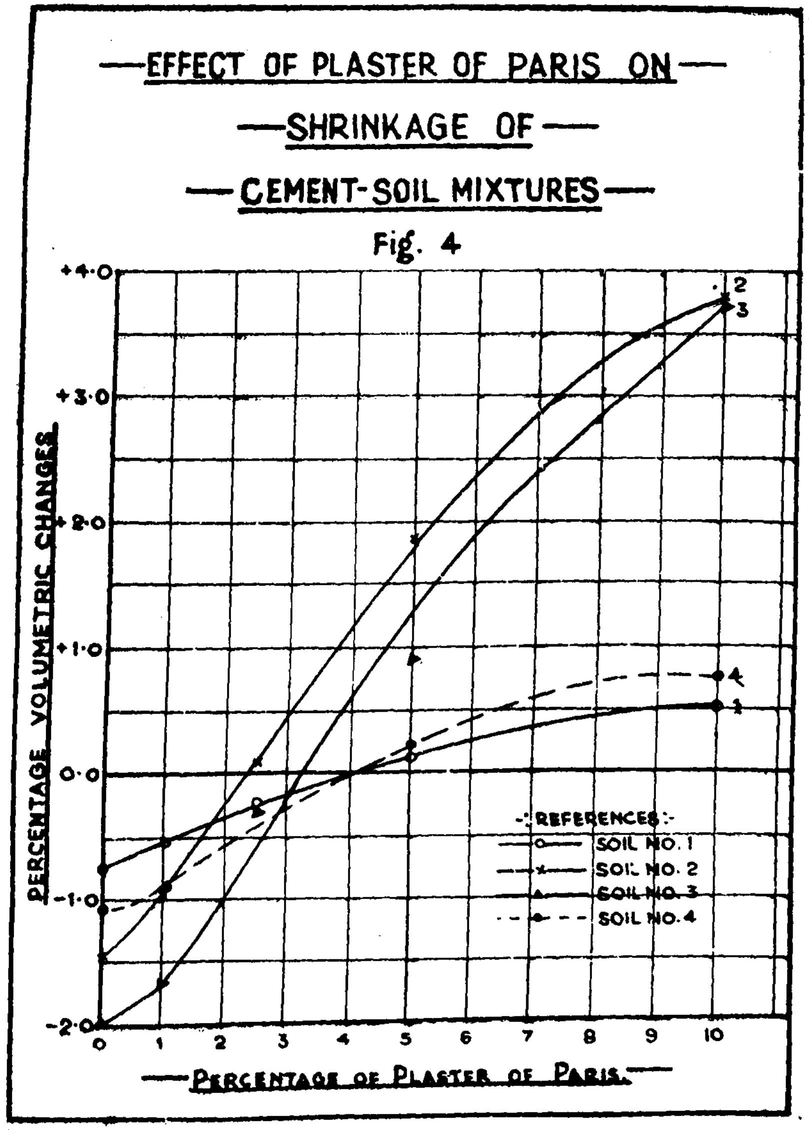
Showing effect of plaster of Paris on the shrinkage of cement-soi mixtures (cement content 2.5 per cent).

Soil	Percentage of Plaster of Paris to cement-soil mixture	Optimum moisture per cent	Density	Volumetric shrinkage per cent
Sandy loam	0	13.0	1.80	-0.72
	1.0	13.0	1.81	-0.56
	2.5	13.0	1.84	-0.24
	5.0	14.0	1.85	+0.09
	10.0	14.0	1.86	+0.52
Silty loam*	0	12.0	1.74	-1.46
	1.0	13.0	1.74	-0.94
	2.5	14.0	1.75	+0.10
	5.0	14.0	1.75	+1.84
	10.0	14.0	1.75	+3.82
Silty clay loam	0	12.0	1.73	-1.98
	1.0	12.0	1.72	-1.71
	2.5	12.0	1.74	-0.30
	5.0	13.0	1.74	+0.89
	10.0	14.0	1.70	+3.69
Loam	0	12.0	1.94	-1.04
	1.0	12.0	1.95	-0.88
	2.5	12.0	1.97	-0.31
	5.0	12.0	1.96	+0.18
	10.0	13.0	1.96	+0.74

Note :—minus indicates shrinkage.
plus indicates expansion in volume.

* Liquid Limit	Plasticity index	Sand content
28.40	9.4	11.80

It will appear from Table 5 that the shrinkage decreased as the quantity of plaster of Paris increased and, when added beyond a certain limit, there was an expansion in the volume of the compacted cement-soil mass. The expansion, however, was comparatively smaller in the case of sandy loam and loam soils than in case of silty loam and silty clay loam soils.



It was concluded that by carefully determining the quantity of plaster of Paris, varying from 2 to 2.5 per cent of dry weight of cement-soil mixture, it could be possible to reduce shrinkage considerably and have joints at much longer distances. It may be pointed out here that whereas by addition of plaster of Paris the shrinkage can possibly be completely cut out, the expansion and contraction of the soil under cycles of wetting and drying will still require provision of joints though at longer intervals. (This aspect of soil behaviour is under study and will form the subject of a separate report).

THE EFFECT OF THE ADDITION OF PLASTER OF PARIS ON OTHER PROPERTIES OF SOILS

The effect of the addition of plaster of Paris on other properties of soils had to be determined to ensure that reduction in shrinkage was not being attained at the cost of other structural properties of soils such as compressive strength and resistance to weathering.

Blocks of 2.5 per cent cement-soil mixtures were therefore prepared with increasing percentage of plaster of Paris. These were cured under wet sand for a week and then allowed to dry in the shade before finally drying them in the oven at 50 deg. C. These were then subjected to compressive strength test. The size of the block was the same as in previous experiments, viz., 2.5 in. diameter and 1.3 in. height. The results are given in Table 6.

It will appear from Table 6 that in the case of sandy loam soil (No. 1) the compressive strength of 2.5 per cent cement-soil mixture increased as the quantity of plaster of Paris increased, but in case of loam soil (No. 4) maximum increase in compressive strength was attained with 2.5 per cent addition of plaster of Paris and further addition beyond that did not result in increased compressive strength whereas in case of silty loam and silty clay loam (Nos. 2 and 3) the maximum increase in compressive strength was with 1 per cent plaster of Paris, and further addition lowered the compressive strength of the resultant mass.

WEATHERING EFFECT

The effect of the addition of plaster of Paris on the behaviour of the cement soil mass under repeated cycles of wetting and drying was then tested. Blocks made of 2.5 per cent cement-soil mixture with increasing percentage of plaster of Paris were subjected to wetting and drying tests. The results obtained are given in Table 7.

Table 6.

Showing effect of plaster of Paris on the compressive strength of cement-soil mixtures.

Soil	Quantity of plaster of Paris in 2.5 per cent cement-soil mixture	Density	Compressive strength lb/sq. in.	
			Initial crack	Final crack
1. Sandy loam	0	1.81	534.04	692.90
	1.0	1.82	672.00	851.20
	2.5	1.84	806.30	1063.12
	5.0	1.84	1008.00	1267.84
	10.0	1.85	1590.06	1800.96
2. Silty loam	0	1.76	549.54	877.25
	1.0	1.76	689.92	1025.92
	2.5	1.75	524.16	931.84
	5.0	1.71	734.72	941.00
	10.0	1.68	667.52	837.50
3. Silty clay loam	0	1.75	836.26	1080.90
	1.0	1.77	1011.76	1352 90
	2.5	1.74	1008.00	1209.60
	5.0	1.71	904.96	1030.40
	10.0	1.65	702.84	806.40
4. Loam	0	1.96	749.00	103[illegible].40
	1.0	1.97	1120.00	1272.32
	2.5	1.95	940.80	1550.00
	5.0	1.95	1120.00	1505.28
	10.0	1.95	1120.00	1594.88

Table 7.

Showing loss of weight of cement-soil mixture with increasing percentage of plaster of Paris during process of wetting and drying.

Serial No.	Description	Per cent cement	Per cent plaster of Paris	Density	PERCENTAGE CHANGES IN WEIGHT DURING WETTING AND DRYING — Number of Cycles: 1	2	3	4	5	6	7	8	9	10	11	12
1.	Soil No. 1 Sandy loam	2.5	1.0	1.81	0.0	0.0	0.0	0.6	0.6	0.6	0.7	0.83	Blocks developed cracks			
			2.5	1.84	0.14	0.14	1.18	1.15	1.15	1.15	1.15	2.65				
			5.0	1.85	0.0	0.2	0.26	1.6	1.6	1.6	1.75	1.95				
			10.0	1.86	0.03	9.03	0.19	2.53	2.53	2.88	3.24	3.84				
2.	Soil No. 2 Silty loam	2.5	1.0	1.74	0.0	0.0	0.19	1.49	1.52	2.29	3.49	5.19	Further operations given up due to high loss in weight.			
			2.5	1.75	0.05	0.05	0.19	1.4	1.4	7.2	18.3	35.3				
			5.0	1.75	0.2	0.2	0.55	2.1	2.45	4.3	8.1	10.0				
			10.0	1.74	0.4	0.4	1.15	3.85	43.65	44.65	45.35	46.75				
3.	Soil No. 3 Silty clay loam	2.5	1.0	1.78	1.0	5.2	19.7	26.2	48.90	65.2			—do—			
			2.5	1.74	14.3	21.7	28.6	40.9	65.9							
			5.0	1.74	1.8	7.2	18.6	26.4	40.55	45.9	59.5					
			10.0	1.70	0.5	0.54	2.2	7.6	25.0	45.2	52.2					
4.	Soil No. 4 Loam		1.0	1.84	0.0	0.56	0.56	0.56	0.56	0.67	0.67	1.00	1.03	1.03	1.23	1.43
			2.5	1.81	0.0	0.84	0.84	0.84	0.84	0.94	0.94	1.21	1.21	1.21	1.31	1.31
			5.0	1.78	0.1	0.60	0.60	0.60	0.60	0.60	0.60	0.80	0.80	0.80	0.80	0.80
			10.0	1.71	0.0	0.80	0.80	0.80	0.80	0.80	0.80	1.07	1.07	1.07	1.07	1.07

Table 8.

Showing volumetric changes of 2.5 per cent cement-loam soil with increasing percentage of plaster of Paris during process of wetting and drying.

Serial No.	Description	Per cent cement	Per cent plaster of Paris	Density	Per cent change in volume after 7 days curing	Per cent volume after drying		1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	12th
1.	Soil No. 4	2.5	1.0	1.84	+0.14	0.88	Vol. Wet	0.0	0.0	0.0	0.09	0.045	0.18	0.045	0.09	0.0	0.0	0.0	+0.09
							Vol. Dry	0.27	0.27	0.45	0.36	0.36	0.45	0.45	0.45	0.45	0.36	0.28	0.40
2.	—do—	2.5	3.5	1.81	+0.39	0.27	Vol. Wet	0.315	0.225	+0.18	.09	+0.09	0.09	+0.09	0.0	0.09	0.09	+0.13	+0.18
							Vol. Dry	0.18	0.270	0.27	0.27	0.27	0.09	0.18	0.18	0.18	0.09	0.09	0.09
3.	—do—		5.0	1.78	+0.70	+0.18	Vol. Wet	+0.53	+0.35	+0.25	+0.25	+0.35	+0.35	+0.35	+0.35	0.35	0.44	+0.44	+0.53
								+.044	+0.0	+0.44	+0.88	+0.26	+0.18	+0.18	+0.18	0.18	+0.18	+0.26	+0.26
4.	—do—		10.0	1.71	+1.3	+0.74	Vol. Wet	+1.23	+1.07	+1.15	+1.07	+1.15	+1.07	+0.98	+0.90	+0.98	1.15	+1.07	+1.07
							Vol. Dry	+0.76	+.579	+.656	+0.82	+0.74	+0.82	+0.74	+0.82	+0.82	+0.94	+0.98	+0.98

Note:—The + sign shows increase in volume or expansion remarks where figures without sign show shrinkage.

It will appear from Table 7 that when plaster of Paris was added in increasing quantities to 2.5 per cent cement-sandy-loam-soil mixture, the resultant compacted mass developed cracks after 8 cycles of wetting and drying and, therefore, made it unfit for permanent structures, in spite of a high compressive strength. Also that plaster of Paris decreased the water resisting qualities of cement silty loam soil mixture to some extent, but in the case of silty clay loam soil it cut down the resistance rather drastically.

The only soil where plaster of Paris can possibly be successfully used without decreasing in any way the water resisting quality is loam soil. Therefore better results could possibly be obtained with other cement-soil mixtures if they were blended with loam. The volumetric changes brought about during increasing cycles of wetting and drying with an increasing concentration of plaster of Paris in a 2.5 per cent cement loam soil mixture is given in Table 8.

The effect of plaster of Paris on the compressive strength of a cement-soil mixture under repeated cycles of wetting and drying was next determined. The results obtained initially and at the end of 12 cycles are given in Table 9.

Table 9.

Effect of plaster of Paris on the compressive strength of cement-soil mixture after weathering.

Soil	Description	Per cent plaster of Paris	Compressive Strength lb/sq. inch			
			Without cycles		After 12 cycles	
			Initial Crack	Final failure	Initial crack	Final failure
No. 4	Loam soil (with 2.5 per cent cement)	0.0	749	1030	1288	1701
		1.0	1120	1272	892	1554
		2.5	940.8	1550	1260	1470
		5.0	1120	1505	1428	1512
		10.0	1120	1594.88	840	1176

It will appear from Table 9 that the compressive strength was not affected by wetting and drying cycles. High concentration of plaster of Paris as 5 and 10 per cent are unsuitable, not only because it does not increase the compressive strength of the resultant mass, but also because it brings about expansion in volume.

SUMMARY

The effect of moisture of compaction on the shrinkage of compacted soils on drying has been studied. It has been shown that shrinkage can be partly reduced by addition of cement in a small quantity, varying with the soil. Shrinkage was also further reduced in case of loam soil without affecting the other structural qualities of the soil, when compacted with small percentages of plaster of Paris. The importance of provision of joints in structures made of cement-soil mixtures has been brought out by the experiments.

Reference:

Standards for Testing Soils (Tentative) 1948— Central Board of Irrigation Pub. 42 Ind.

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INFORMATION SERVICE

1. STABILIZATION OF GUMBOTIL SOIL FOR HIGHWAY USE.

By O. H. Patel.

[The following article is extracted from a Thesis submitted by Shri O. H. Patel, based on his research work in the U. S. A. It treats of the problem of stabilization of gumbotil soil, a soil which is the nearest approach in America, to the Indian black cotton soil. It is hoped the information given in this article would be of help to those engineers who have to deal with black cotton soil problems in this country.—Ed.]

OBJECT OF THIS STUDY

A typical heavy plastic clay soil called "black cotton soil" and commonly known as "regur" occurs in large tracts of India. In the tablelands to the south of the Vindhya Hills, in the whole of the Deccan Plateau, and over a considerable portion of South India, a large percentage of the arable soil is black cotton soil. The problem of road making on this soil has attracted the attention of many highway engineers because traditional methods of construction involve very high costs and low cost methods of stabilization of this soil have so far not been successfully developed. The soil presents special difficulties because of its peculiar nature, (1) excessive swell after absorption of moisture in the monsoon season when it becomes highly unstable, and (2) the formation of wide shrinkage cracks after drying.

A similar type of soil with similar characteristics exists in parts of Iowa, in the United States of America. This soil is commonly known as "gumbotil" and it is of glacial origin. Gumbotil is a highly plastic soil undergoing great volume changes and loss of stability with the absorption of water. For this reason it creates problems of a type similar to those presented by the black cotton soil of India. Hence, the object of this study was to develop methods of improving the properties of gumbotil soil as indicated by laboratory tests. The investigations made in this study included a careful and complete analysis and the determination of the effects of various stabilizing agents on gumbotil soil which may serve as an indication of the effectiveness of similar treatments of black cotton soil.

OUTLINE OF THE PROBLEM

The properties of gumbotil soil were investigated for the specific identification of the mineral group present in it. Also, a careful study was made to determine why it exhibits the typical

characteristics of great volume change and high plasticity index. Furthermore, by means of laboratory tests, its value from the structural point of view was determined Various low cost admixtures involving the use of materials such as lime and Portland cement in small percentages were tried to observe the changes in its engineering properties and the improvement in its behaviour under adverse conditions. The effect of these small percentages of admixtures on its water repellant character was also studied.

* * * *

PREVIOUS WORK

A method commonly used to improve clay soils is to increase the angle of internal friction by the addition of large size granular material, while the cohesion can be preserved by water proofing the soil To take advantage of the high cohesive strength of this type of soil by the proper control of its moisture content a bituminous surface treatment using a heavy road oil was used by Minnesota on a section of road in the Northwestern part of the State This area of Minnesota is very flat, being the bed of an old glacial lake. Gravel had to be shipped long distances and the road became slippery and greasy in a very short time, even after a heavy coat of gravel was placed on it. This treatment of gravel was not successful, and for this reason the bituminous treatment method was given a trial in 1925.

* * * *

A typical analysis of the road oil used to stabilise this gumbo is as follows :—

Specific gravity	...	1·044
Flash point	...	164° C.
Specific viscosity at 60° C.	...	12·9
100 penetration residue	...	65·2
Ductility of residue	...	100+
Loss at 163° C.	...	2·8
Total bitumen soluble in carbon di-sulphide	...	99·91
Total bitumen insoluble in 86 deg. naptha	...	16·23

After treating this soil with the road oil just described, it was found that the soil could stand up much better than by any other method of treatment tried before, even for traffic intensity of 705 vehicles per day. This treatment pointed the way to a new field of research for treatment of heavy clays.

* * * *

Dr. Hans F. Wintercorn, one of the leading investigators observed that stabilization of heavy clay is a function of the general character of the soil and its exchange cations. The exchange cations of high valency, large ionic radii, and low water affinity are desirable. It was also shown in the extensive studies by the Portland Cement Association, Endersby and Becker and Benson that the successful treatment of heavy clay soils for soil cement construction required about 16 20 per cent of cement and for soil-bitumen treatment about 12-16 per cent of bitumen for meeting the prescribed specifications. These high percentages of cement and of bitumen were not considered to be economical and hence, these treatments have not been generally used by highway departments.

Further experiments with heavy clay soils, were carried out to determine the effect of the addition of small amount of cement. In these tests it was found that the addition of small amount of cement to these soils caused a marked improvement of the plasticity index, the volume change and shrinkage properties and made these soils function satisfactorily as subgrades capable of supporting pavements over them.

* * * *

The most recent experimental work involving the stabilization of clay soils by the addition of lime is that reported by the Texas Highway Department. The addition of small amounts of lime was found to be surprisingly effective in the treatment of the highly plastic subgrade soils of Texas. This treatment increased the stability and water repelling properties of the soil sufficiently to permit its use as a base for higher type of pavement. The strength in the confined and unconfined compression test, the shrinkage and swelling properties, and the plastic properties were greatly improved. The results of these studies also indicated that the favourable effect of lime upon clay soils is not of short life, and the following general conclusions were indicated by them:

1. Soil-lime stabilization has a definite application in highway construction for improvement of certain subgrade and flexible base materials.

2. Many natural soils are suited to lime stabilisation. The identical materials proposed for use should be subjected to preliminary physical tests.

3. Good proportioning and mixing of constituents are advantageous.

4. The compacting moisture should be at or slightly below optimum moisture content for the given compactive effort employed.

5. A high degree of compaction greatly improves the stability of soil-lime mixtures.

6. Suitable curing procedures of the soil-lime mixtures are also important to develop high strengths.

7. Application of a bituminous wearing surface is desirable.

PRESENT INVESTIGATION

The whole of the present study is divided into two parts. The first deals with the study of the properties of the untreated soil, and the second part deals with the study of the treated soil. The methods of procedure for the particular tests used in this study are described, and the results are then presented and discussed.

1. Test Procedure:

Routine tests on samples were performed in the soils laboratory of the Engineering Experiment Station. These tests in most cases conformed with the specifications set up by the American Society for Testing Materials (A.S.T.M.) and the American Association of State Highway Officials (A.A.S.H.O.). In some cases the standard test procedure was altered and the methods as adopted in these cases are described in detail in this section.

The methods adopted for the determination of the physical properties of gumbotil soil are as follows:

1. **Moisture Content Tests:**

   Field Moisture, Hygroscopic Moisture, Centrifuge Moisture, equivalent (C.M.E.) and Field Moisture equivalent (F.M.E.) ......A.S.T.M. D-426, D-425, A.A.S.H.O. Standard T-93 and T-94.

2. **Mechanical Analysis:**

   ......A.A.S.H.O. Standard T-87 and T-88 A.S.T.M. D-421, D-422.

3. **Specific Gravity:**

   Pycnometer method. A.A.S.H.O. Standard T-100.

4. **Plasticity:**

   Plastic Limit (P.L.), Liquid Limit (L.L.), and Plasticity Index (P.I.). A.A.S.H.O. Standard T-89, T.90, T-91: A.S.T.M. D-423, D-424.

11. **Clay Mineral Identification Test by the Differential Thermal Method.**

The principle involved in the clay mineral identification test by the differential thermal method (Fig. 2) is that the endothermic and exothermic reactions taking place in soil sample between a temperature range of 100 deg. to 1,000 deg. C. are recorded automatically on a photographic paper by differential thermo couple, a galvanometer and a mirror attachment. The furnace in which the test was run was started with the controller set at 6.2 amps and 10 volts. The temperature was raised gradually at the rate of 50 deg. C. for every three minutes from room temperature to 1,000 deg. C. By comparing these thermal curves with the thermal curves of standard known clay minerals, it is possible to identify the clay mineral Two thermal curves were obtained of this soil. The first was on a soil sample passing through a 325-mesh sieve, i.e., the size of the particles was 50 microns and smaller. The second thermal curve was obtained on that part of the soil which had an equivalent diameter of 0.2 micron and smaller. For obtaining the latter sample of soil, a Sharples Cut off Centrifuge was used to separate and remove particles larger than 0.2 microns.

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STABILIZATION OF GUMBOTIL SOIL

Since gumbotil soil is highly plastic and has very low stability at normal field moisture contents due to its fine sized particles in the deflocculated state, it was thought desirable to add small percentages of lime, cement, barium hydroxide and calcium chloride and to study the change in its properties for each of these admixtures. Lime was obtained from the Chemistry Laboratory. It was a pure calcium oxide in the dehydrated state at the time of use. The cement used was regular Portland cement. Barium hydroxide was obtained also from the Chemistry Department and was in pure crystalline state. The calcium chloride used was the flake type and also was in the dehydrated state. Before mixing with the soil, all these chemicals were ground to fine powder passing through sieve No. 100. It was expected that lime and cement, due to their flocculating properties, might give satisfactory results.

Various test procedures were adapted to measure the stabilizing effect of the gumbotil soil with the admixture as follows:—

1. **Mechanical Analysis:** An amount of lime and cement equal to four per cent by weight of the oven dry soil was added

separately to the raw soil passing a No. 10 sieve. Both the cement and the lime and soil were thoroughly mixed and samples were prepared in a standard proctor mould. After allowing it to dry for two days, it was pulverised with a rubber hammer and the pulverised mixture was passed through sieve No 10. The mechanical analysis was carried out on this modified soil (A.S.T.M. D-421). The gradation of these modified soil samples are presented in Table 1 and in Fig. 3.

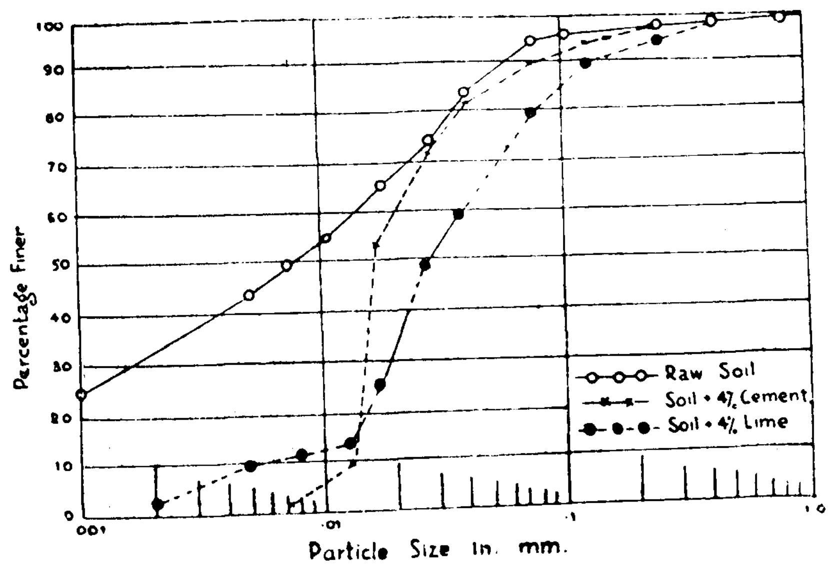


Fig. 3. Particle Size Distribution of Modified Soils.

2. **Plasticity Tests:** Pure calcium oxide (lime) as obtained from the Chemistry Laboratory was pulverised to pass through sieve No. 100 and added to soil passing sieve No. 40 in amounts equal to 1, 2, 3, 4, 6, and 8 per cent of the oven dry weight of soil. Plastic limit and liquid limit tests were carried out for the following two cases:

1. Tests carried out immediately after dry mixing the lime and one hour of hydration.

2. Tests carried out after allowing the mixed lime to hydrate for 48 hours.

Similar tests were carried out for both conditions referred to above with the addition of 1, 2, 3, 4, 6, and 8 per cent of regular Portland cement.

Table 1.

Mechanical Analysis of Modified Soils.

Sieve No.	Size (mm)	Percentage finer than the size shown		
		Soil	Soil + 4 per cent cement	Soil + 4 per cent lime
1½ in.	38.1	100	100	100
1 in.	26.7	100	100	100
¾ in.	18.8	100	100	100
½ in.	12.5	100	100	100
⅜ in.	9.4	100	100	100
No. 4	4.7	100	100	100
No. 10	2.0	99.60	99.7	99.7
No. 20	0.83	99.35	99.30	99.3
No. 40	0.42	98.21	98.20	98.2
No. 60	0.25	96.75	96.80	95.2
No. 140	0.125	95.59	95.50	89.6
No. 200	0.074	94.70	89.60	79.4
Size and percentage based on hydrometer readings and time of settling	0.038	85.50	84.00	58.0
	0.027	76.25	74.00	50.0
	0.017	66.20	54.00	28.0
	0.013	55.25	10.00	14.0
	0.0075	50.00	—	12.0
	0.0051	40.30	—	10.0
	0.0020	—	—	2.0
	0.0010	25.50	—	—

3. **Shrinkage Limit Tests:** It was found from the P. I. tests that hydration of cement or lime takes place within one hour, so it was decided to add the desired amount of lime or cement to the soil passing sieve No. 40 and carry on the tests for one soil sample only in accordance with the Standard Specification A. S. T. M. D-427. Amounts of (1) lime equal to 2 and 4 per cent,

(2) cement equal to 2 and 4 per cent, and (3) calcium chloride equal to 4 per cent of the oven dry weight of the soil, were added and thoroughly mixed with the soil. All these chemicals were thoroughly pulverised to pass a No. 100 sieve before they were added to the soil. Shrinkage tests according to A. S. T. M. D-427 were carried out.

4. **Standard Proctor Density and Optimum Moisture Contents Tests:** Lime, cement, calcium chloride, and barium hydroxide in the desired percentage of oven dry soil were intimately hand mixed dry after thoroughly pulversing the former. A granular mixture of soil, sand, and gravel was also prepared as per grading curve given in Fig. 4. Maximum standard proctor density and the optimum moisture content for each of the above mixtures were found as indicated by the results of these tests. The procedure adopted was in accordance with the A. A. S. H. O. Standard T-99.

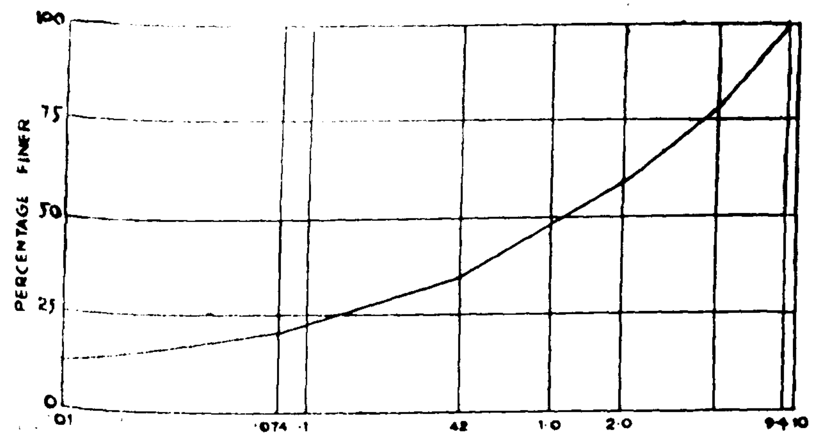


Fig. 4. Gradation for Granular Mixture Using Coarse and Fine Sand and Gumbotil Soil as the Binder.

5. **California Bearing Ratio and Expansion Tests:** Modification was made in this test which is normally carried out at the modified proctor density and the corresponding optimum moisture. Samples for these tests were moulded at standard proctor density and optimum moisture content as found out by test No. 4 just described Desired amounts of chemicals, i. e., lime, calcium chloride, barium hydroxide, were dry mixed and cement, allowed to hydrate for about two hours at the predetermined moisture content. The loads at various penetrations and the swell after soaking were tabulated.

RESULTS OF THE TESTS OF THE MODIFIED GUMBOTIL SOIL

(i) **Mechanical Analysis:** The particle size distribution of the modified soil when mixed with 4 per cent of lime and with 4 per cent of cement is given in Table 1 and in Fig. 3.

Table 2.
Shrinkage Limit Values of the Modified Soil.

Test No.	Description of Admixture	Shrinkage Limit	Shrinkage Ratio	Volumetric Shrinkage	Linear Shrinkage
		per cent		per cent	per cent
1	Raw Soil	9.58	2.10	59.5	14.3
2	Soil+2% cement	13.00	1.95	48.7	12.5
3	Soil+4% cement	21.90	1.66	26.7	7.5
4	Soil+2% lime	24.00	1.61	22.5	6.5
5	Soil+4% lime	29.00	1.45	13.0	4.0
6	Soil+4% calcium chloride	9.88	2.06	57.5	13.5

(ii) **Stand Proctor Densities:** The results of the proctor density tests of the modified soil are given in Table 3 and Fig. 5.

Table 3.
Standard Proctor Density Values and Optimum Moisture Content of Modified Soil.

Test No	Description of Admixture	Standard Proctor Dry Density lb per cu. ft.	Optimum Moisture Content Per cent
1	Raw Soil	97.5	26.3
2	Soil+4 per cent cement	96.5	23
3	Soil+2 per cent lime	96.25	24.8
4	Soil+4 per cent lime	92.8	26
5	Soil+4 per cent barium hydroxide	94.0	27
6	Soil+4 per cent calcium chloride	106.0	18
7	Mechanical mixture	130.0	10

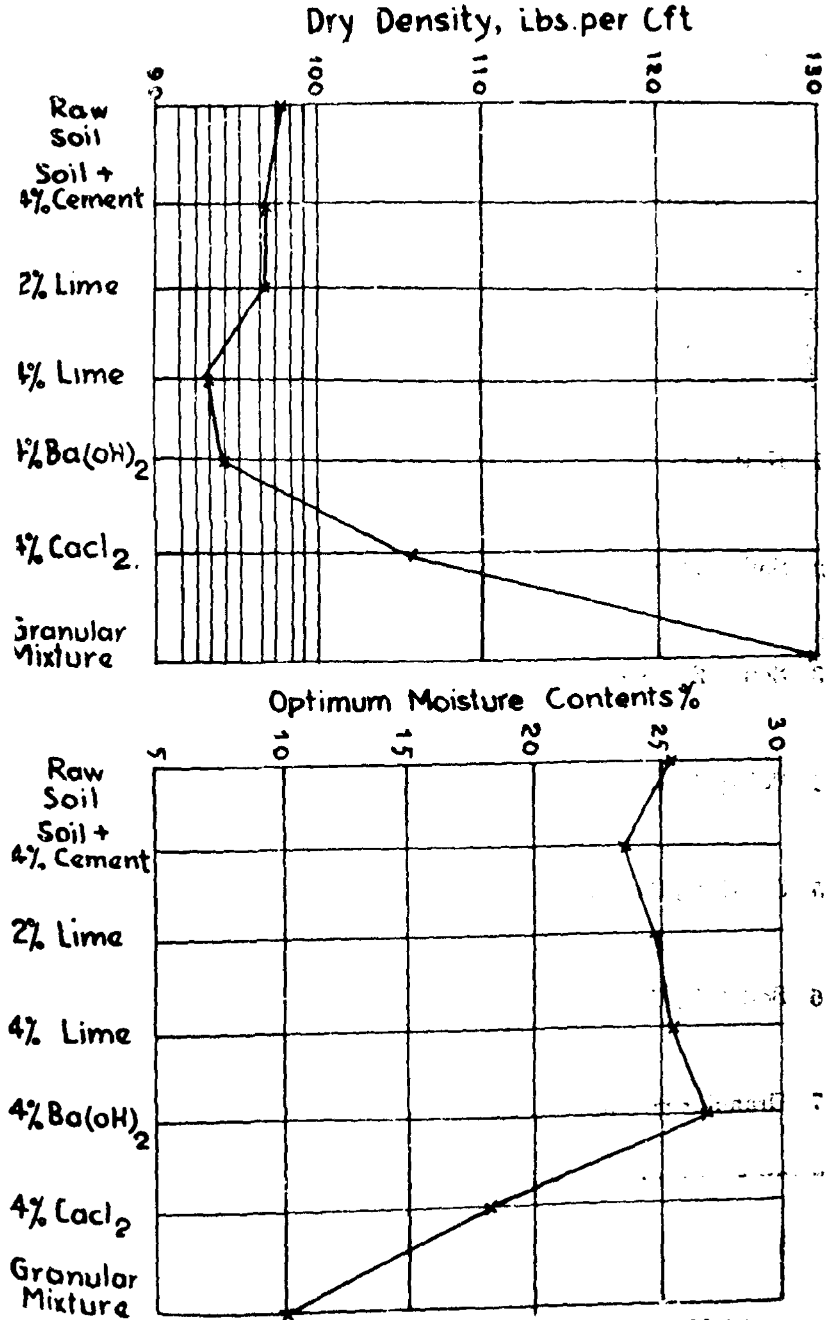


Fig. 5. Standard Proctor Densities and Optimum Moisture Contents for Modified Soils as Compared with Raw Soil.

The results of the expansion or swell tests after four day soaking of the modified soil using various admixtures are given in Table 5 and in Fig 7. The moisture contents in the top, middle, and bottom layers of the soaked specimens of the modified soil are given in Table 6.

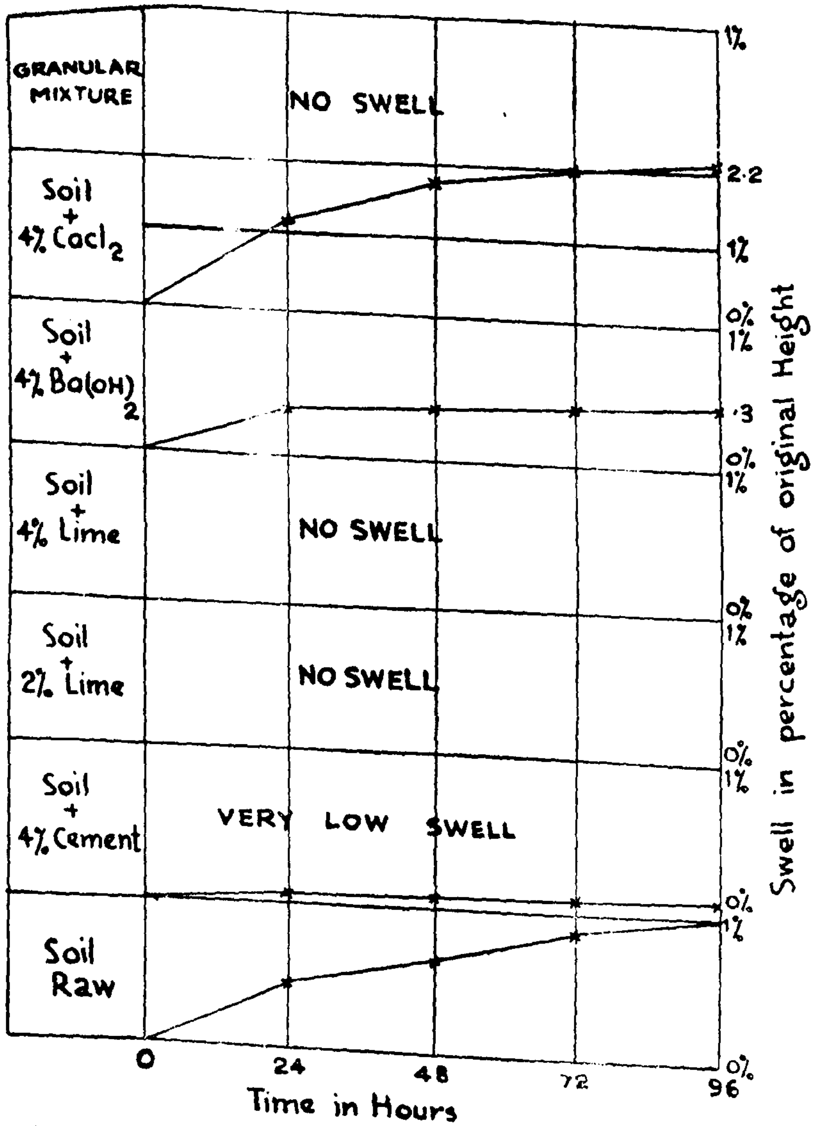


Fig. 7. Swell of Modified Soils in California Bearing Ratio Test at Standard Proctor Density.

Table 5.

Expansion or swell Expressed as Percentage of Original Heights for Modified Soils.

Test No.	Soil Mixture	Expansion in Per cent of original height for various periods of soaking			
		24 hrs.	48 hrs.	72 hrs.	96 hrs.
		Per cent	Per cent	Per cent	Per cent
1	Natural Soil	0.40	0.60	0.80	1.00
2	Soil + 4% cement	0.02	0.04	0.06	0.08
3	Soil + 2% lime	0.0	0.0	0.0	0.0
4	Soil + 4% lime	0.0	0.0	0.0	0.0
5	Soil + 4% $Ba(OH)_2$	0.30	0.30	0.30	0.30
6	Soil + 4% $CaCl_2$	1.20	1.85	2.15	2.20
7	Mechanical mixture of soil with sand and gravel	0.0	0.0	0.0	0.0

Table 6.

Moisture Contents in the Three Layers, Top, Middle, and Bottom, of the Soaked Specimen after Four Days of Soaking of the Modified Soil using various Admixture.

Test No.	Soil Mixture	Top layer (water directly over it)	Middle layer	Bottom layer (water directly beneath it)
		Per cent	Per cent	Per cent
1	Natural Soil	33.6	29.5	32.0
2	Soil + 4% cement	31.4	31.2	31.4
3	Soil + 2% lime	27.0	27.0	26.9
4	Soil + 4% lime	34.2	32.2	33.0
5	Soil + 4% $Ba(OH)_2$	27.7	27.0	27.6
6	Soil + 4% $CaCl_2$	30.7	18.5	25.0
7	Mechanical mixture	10.3	10.0	10.8

DISCUSSION OF RESULTS

A. Identification of Clay Minerals by Differential Thermal Methods

Recent investigations have established the fact that certain clay mineral groups in clay soils are responsible for the properties and behaviour observed for these soils in which the clay minerals are present. The identification of the clay mineral group is, therefore, very helpful in determining the behaviour of the soil and the type of treatment required to correct objectionable features of the soil. Dr. Johnson prepared synthetic mixtures with two different mineral groups and found out that these materials exhibited a wide range of properties. The thermal analysis is one of the best methods used today for the identification of the clay minera's group. Thermal curves on the kaolinite and montmorillonite are shown in Figs. 8 and 9. These were obtained on material passing through a 325-mesh screen. The endothermic reaction at 590 deg C. and the exothermic peak at 975 deg. C. shown in Fig. 8 is typical of the clay mineral kaolinite. The area under the endothermic curve has been used as a quantitative measure of the amount

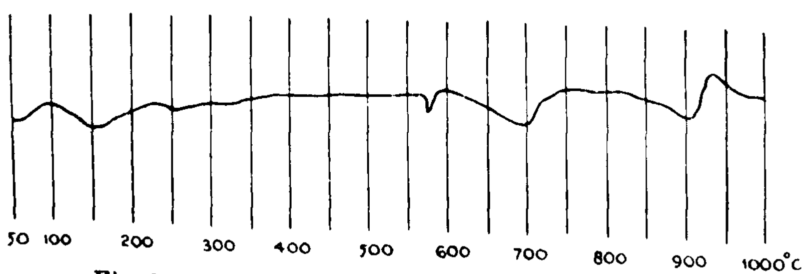


Fig. 8. Thermal Curve for Montmorillonite Sample.

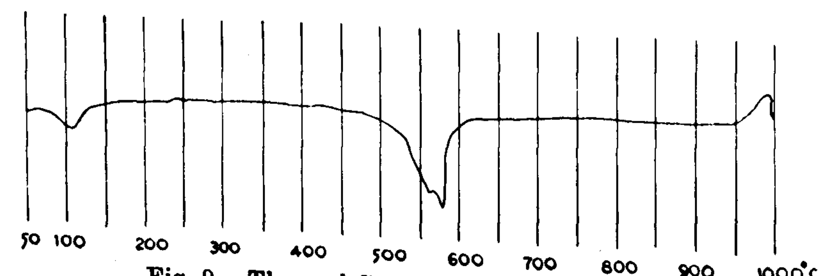


Fig. 9. Thermal Curve for Kaolinite Sample.

of kaolinite present in the given sample. The characteristic endothermic peaks for montmorillonite occur at the following temperatures, an initial peak in the range of 100 to 250 deg. C., a second peak between 600 to 700 deg. C., and a third peak at about 900 degrees C. The exothermic peak occurs at about 975 deg. C.

The thermal curve for the gumbotil soil sample passing through a 325-mesh screen is given in Fig. 10 This curve did not provide reactions similar to any standard curve of known clay minerals.

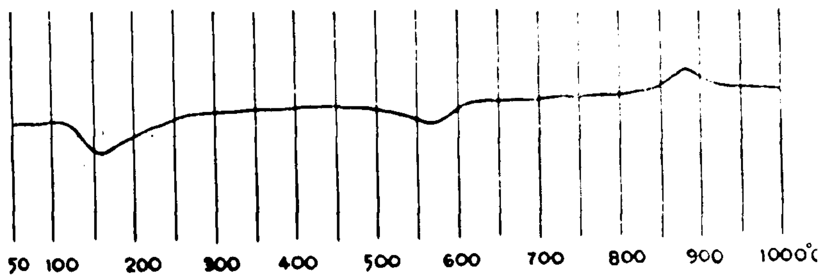


Fig. 10. Thermal Curve for Gumbotil Soil. (Max. Size 43 Micron).

It gave two endothermic reactions, one at 150 deg C. and the other at 550 deg. C. There was also an exothermic peak at 875 deg. C. It was assumed that the presence of some other unknown materials changed the pattern of the curve with the result that the clay minerals in the soils could not be identified. To get a clearer picture of the mineral group, the particles of 0 2 micron size and below were separated by Sharples Super Centrifuge The thermal curve obtained using the sample is given in Fig 11. It gives four thermal reactions as follows :—

Endothermic peak at 150 deg. C.
Endothermic peak at 575 deg. C.
Endothermic peak at 850 deg. C.
Endothermic peak at 900 deg. C.

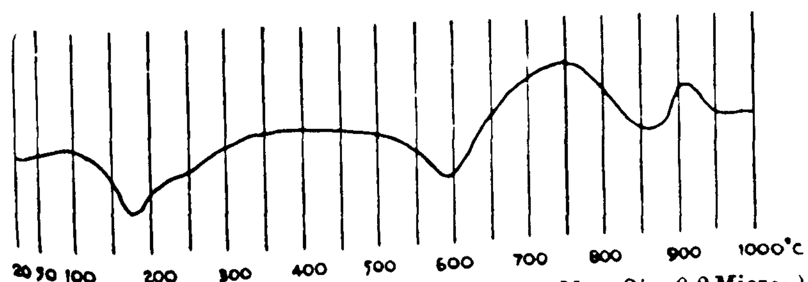


Fig. 11. Thermal Curve for Gumbotil Sample. (Max. Size 0·2 Micron)

This agrees with the standard mineral group montmorillonite except that the reactions are at slightly lower temperatures. This may be due to the presence of Fe-salts which have a tendency to cause the shift to lower temperatures.

B. Mechanical Analysis of the Modified Soils

The changes in the particle size distribution of the modified soil with lime and cement are shown in Fig. 3 and in Table 1. Lime and cement flocculate the small colloidal particles of the soil and form lumps of clay. The colloids are not present in their normal form in the modified soils and hence it is to be expected that the modified soils will not have the same properties as the natural soil Practically all the colloids and clay particles are flocculated to form particles approximately of silt size. Both lime and cement have approximately the same flocculating effect.

C. Plasticity Index

The plasticity properties of the modified soils were compared with those of the natural soil. The results are highly significant considering the small percentages of lime and cement added. The tests were carried out after two hours of hydration and 48 hours of hydration. There was little difference in the results although a greater reduction in the plasticity index was obtained for the lime than for the cement. The soils with the longer period of hydration gave slightly better results than for two hours of hydration.

The plasticity index of the raw soil was 41 and was gradually reduced to 3·5 with 8 per cent of cement and to 2·0 with 8 per cent of lime. This reduction in the plasticity index is a definite evidence that the cohesion of the soil particles in the pulverised mixtures is materially reduced and approaches the values found for sandy and silty soils. The marked increase of the plastic limit for small additions of lime to the soil is a definite indication of the advantage of lime to improve heavy clay soils. It was seen that 2 per cent of lime had a slight advantage over 4 per cent of cement in this soil.

D. Shrinhage Tests

The results of experiments on volumetric changes also show that lime provides even a greater reduction in volume change than cement. It was seen that with this soil in this respect also 2 per cent of lime provided a greater improvement of the soil than 4 per cent of cement.

It was found that calcium chloride provided no improvement in the plasticity properties of this soil. It neither increases the shrinkage limit nor reduces the volume change.

E. Density Tests

The dry densities with lime decreased with an increase in lime content while the optium moisture cantents remained nearly the same. The mixtures of 4 per cent barium hydroxide and gumbotil soil resembled the one made with the addition of lime in both the densities and moisture contents. The 4 per cent calcium chloride mixture gave a maximum density of 106 pounds per cubic foot compared to 97·5 pounds per cubic foot of raw soil. This was expected because calcium chloride increases the surface tension of the water and thus improves the bounding of the clay particles in the mix. A granular mixture prepared by the addition of fine and coarse sand gave a density of 130 pounds per cubic foot at an optimum moisture content of 10 per cent. This high density and low moisture content is the normal result for a well-graded granular mixture.

The addition of 4 per cent cement gave C.B.R. values, at 0·1 inch penetration, of 9·3 for the unsoaked condition and this was increased to 15·0 after soaking. The result is certainly most encouraging because the soil is greatly improved and behaves better in the saturated condition. The results observed with lime were even more satisfactory than the results with cement. With the addition of 4 per cent lime, the C.B R. values at 0·1 inch penetration increased from 8·3 to 20·0 per cent for the unsoaked as compared to the soaked condition showing the marked advantage of lime in providing stabilization. It was further observed that 2 per cent lime gave still better results than for the mixture with 4 per cent lime The two per cent lime admixtures increased the C.B.R values from 17·0 to 45·0 for 0·1 penetration when compared with the C.B.R. value for the 4 per cent mixture. The reason why the two per cent lime mixture gave better results than the 4 per cent lime mixture is not readily explained. However, it is possible that there may be some optimum lime content based on the exchange capacity of the soil which may give the best results in the C.B.R. and swell tests. The results of the tests in this investigation indicate that 2 per cent admixture of lime is close to this optimum value.

Since lime gave exceptionally good results when used with gumbotil soil, it was thought desirable to see how barium hydroxide, which is an alkaline material somewhat similar to lime, would change the properties of this soil. It was found that barium hydroxide did not improve this soil. Similarly, when 4 per cent of calcium chloride was added, it was found that it helped to improve the California Bearing Ratio in the unsoaked condition but after soaking, the soil was as poor as it had been without calcium chloride.

A granular mixture consisting of gumbotil soil as a binder and fine and coarse aggregate to provide the grading shown in Fig. 4, when compacted at standard proctor density and optimum moisture content did not provide satisfactory results in the C.B.R. and swell tests, although the density was very high due to addition of fine and coarse sand. So it may be said that sand and coarse aggregate cannot be depended upon to improve the strength of the gumbotil soil.

The results of the expansion tests (Table 5 and Fig. 7) after four days of soaking showed that calcium chloride provided the greatest expansion of all the methods of treatment used in this study. The swell was practically reduced to zero when lime and cement were used as an admixture and this was a definite advantage. This may be explained by the fact that calcium chloride deflocculate this soil while lime and cement flocculate it.

CONCLUSIONS

1. There seems to be need for further investigation of the advantages provided by using lime, a very cheap material, for stabilization of plastic clays The exact nature of the reactions are not known. It should be possible to find out, in general, the class of clays and clay minerals that can be effectively stabilized with lime. There may be some relation between the clay mineral group, its chemical composition, and the type and amount of lime to be used.

2. Quantitative studies should be made to find out the exact amount of lime necessary to obtain the best results in stabilizing plastic clays and other soils. This amount may be related to the base exchange capacity of the soil and its chemical composition. Also, the application of lime in amounts less than the exchange capacity of the soil may reveal some important facts.

II. A NOTE ON THE USE OF "MOORAM" SOIL FOR BASE COURSE FOR MACADAM ROADS.

BY

G. SRIRANGACHARY.

During experiments conducted at the Soil Research Laboratory Highways Department, Madras the following interesting results were noted:

(A) 1. "Mooram" may not be such a good soil as is generally believed. Blind faith in any available mooram as good enough for base course, had resulted in the past in costly failures over clay subgrades.

The plasticity index is usually very high, especially in the mooram occurring below or nearby clay soils; vide analysis of some mooram soils in Indian Roads Congress Volume XIV-I, pages 126 to 135.

A very good "mooram" occurring near Pallavaram Hills (near Madras) has a plasticity index-15 per cent and the fraction passing No. 10 mesh was only 48 per cent.

Mooram occurring in black cotton soil areas near Guntur has a plasticity index of 32 per cent to 42 per cent.

2. The grading is generally poor, due to deficiency in coarse sand and fine sand fractions. Even with a good mooram (e. g., from pallavaram quarry near Madras) it is difficult to obtain density exceeding 120 lb/cu. ft.

3. Even a good mooram bed after it is compacted to maximum density, if allowed to soak in water may lose about 60 per cent or more, of its bearing power.

4. The so-called "screened-mooram" occurring in some specifications may often be worse than the unscreened mooram, when used as a base course.

5. The addition to the mooram of the missing sand fractions, makes the mixture an ideal soil.

For instance the good mooram from Pallavaram Quarry when mixed with coarse and fine sand available nearby showed the following results :—

	Plain Mooram Pallavaram Quarry	Mooram+Coarse sand + Fine sand (Proportion 7 : 4 : 4).
1. Per cent passing No. 10 sieve.	48 per cent	68.5 per cent
2. Plasticity Index	15 per cent	5 8 per cent
3. Proctor Maximum Density	118 lb/cu. ft	133.5 lb cu. ft
4. Cone bearing strength immediately after compaction	700 lb/Sq. in. (CBR-16.0 approx.)	950 lb/Sq. in. (CBR-22.5 approx.)
5. —Do—after soaking in water for 3 days	200 lb/sq. in. (CBR-5.5 approx.)	650 lb/sq. in. (CBR-15.5 approx.)

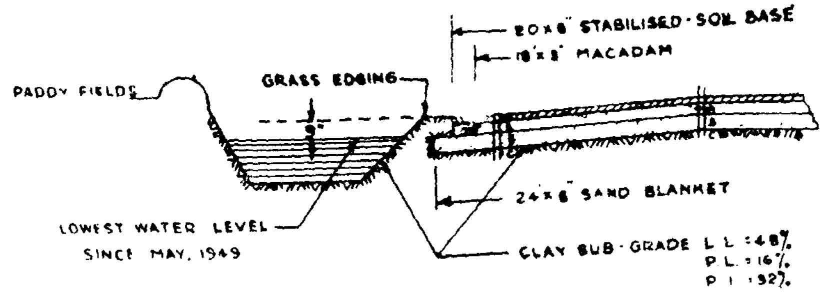
Another instance was a mooram available in a quarry 9 mile south of Guntur.

	Plain Mooram from quarry at mile 9 of Guntur-Peddanandipad Road.	Mooram + a local sand (Proportion 1 : 3)
1. Per cent passing No 10 sieve	35 per cent	60 per cent
2. Plasticity Index.	30 per cent	3.5 per cent
3 Proctor Maximum Density	115 lb/cu. ft	125 lb/cu. ft
4. Cone bearing strength immediately after compaction	350 lb/sq. in.	420 lb/sq. in.
5. —Do—after soaking in water for 3 days	150 lb/sq. in.	300 lb/sq. in.

(B) The importance of the bearing test after soaking in water for three days, has been amply demonstrated by the results observed in a water logged area of very poor drainage near Madras.

Here the clay soil subgrade has a Plasticity Index = 32 per cent and Liquid Limit = 48 per cent

The water table is only 9 inches below road level. The moisture in the clay subgrade and stabilised soil base course after four months continuous soaking was observed to be as noted below:



Moisture content in	Near the edge. per cent	At the Centre. per cent
Base course—A	6.0	4·0
Blanket—B	8.0	5·0
Clay Sub-grade—C	32.0	27.0

Many time during and after rains, the water level came within three inches of the macadam. The base course which was designed for a maximum absorption of 10.5 per cent in a free condition in the laboratory absorbed a maximum of 8 per cent water under actual field conditions.

(C) Based on experiments in the Laboratory, and also from experience of full scale field experiments, the following controls are now specified by the Madras Soil Research Laboratory :—

CONE PENETRATION BEARING VALUES.

	Sub-grade	Base Course or sub-Base.
1. Base course for Macadam road immediately after compaction.	lb/sq. in.	lb/sq. in.
After soaking in water for 3 days	250 to 180	450 to 400.
	— —	350 to 300.
2. Sub-base for cement concrete Road immediately after compaction.	250 to 180	1000 to 800.
After soaking in water for 3 days	— —	600 to 500.

(D) If laboratory tests should show that even the best mixture of local soils falls short of the required power, stablised soil for base course is not recommended.

Experience in Madras shows that in a given locality different soils to make up deficiencies in each other may be expected to be available nearby; and it only requires patient search to locate them. Even in the worst black cotton soil area (near Guntur) we could locate within a distance of 6 miles enough sand to mix with the Mooram, [See A (5) above.].

(E) Tests on clay subgrades (in existing roads) in soaked condition show:

1. Clay soil subgrade of old roads is generally found to be already compacted by traffic to a density equal to or a little over Proctor Maximum Density. This density is usually 95 to 100 lb/cu. ft. At this density the subgrade is likely to absorb water up to 27 per cent.

2. Bearing tests of soaked clay subgrades show that if the moisture content of the subgrade exceeds plastic limit of the clay + 6 per cent the bearing strength is likely to fall below 180 lb/sq. in. (by the cone penetrometer). Such a condition would indicate the necessity to re-inforce the subgrade either by mixing course sand for the top layer (about 6 in. is usually sufficient) or else by interposing a sand blanket course 3 in. to 8 in. thick between the clay sub-grade and the base course.

3. Mixing course sand with black cotton soil (P.I. 42 per cent and L L. 65 per cent), in the ratio of 2 sand to 1 clay, improved the bearing power in fully soaked condition from 50 to 200 lb/sq. in (by cone penetrometer). The density increased from 98 lb to about 106 lb per cu. ft.

It is however difficult to carry out this kind of mixing on the field over large areas and is likely to be very expensive.

4. Sandy soil or loam which can be compacted to a density of at least 112 lb per cu. ft. may be used as a blanket course or sub-base; the thickness of this course will depend on the actual water conditions expected of the subgrade For partial soaked conditions, e.g., on high embankments with low water table, the thickness may be 3 in and for low lying water logged areas the thickness may be 6 in. to 8 in.

(F) Cost analysis is most vital in all cases where stabilised soil is found to be practicable after laboratory tests.

In general, clay soil areas are deficient in stone quarries. The cost of soling stone may then be excessive. Under such conditions, stabilised soil properly designed and constructed may yield striking economies.

The patience and ingenuity of the Soil Engineer will help to solve most problems.

The striking similarities of engineering properties, of mooram in the States of Madras, C.P., and Bihar (See Indian Roads Congress Volume XIV-part I) indicates the possibility of evolving a technique of design and construction which may be applied over large areas of this country.

INTERNATIONAL ASSOCIATION FOR BRIDGES AND STRUCTURAL ENGINEERING

List of the Subjects of the 4th Congress

Cambridge, Great Britain, August 1952 (See notice on P. 206)

(A) General questions.

I. Bases of calculations; safety.

(1) Loading of bridges and structures (Influences of wind, earthquakes, etc.)

(2) Dynamic problems.

(3) Consideration of the actual conditions for deformation (Plasticity, creep, etc.).

(4) General conclusions regarding safety of structures.

II. Development of the methods of calculation.

(1) Analytical methods of the theory of elasticity and Plasticity.

(2) Numerical methods in applied statics.

(3) Other methods of calculation (Approximation methods, relaxation method, calculation regarding rupture, experimental statics, etc.).

(B) Metal structures.

I. Fundamental principles.

(1) High-grade structural steel, light metals.

(2) Welding and welded connections.

II. Practical applications.

(1) Problems in steel building construction.

(2) Structures in light metals.

(3) Special erection methods.

(4) Details of design.

(C) Concrete and reinforced concrete structures.

I. Fundamental principles and the properties of concrete.

(1) Composition of concrete; influence of the preparation, transport and placing on the design of structures.

(2) Properties of concrete, average tensile strengths and their variations.

(3) Effect of repeated and continuous loading, creep.

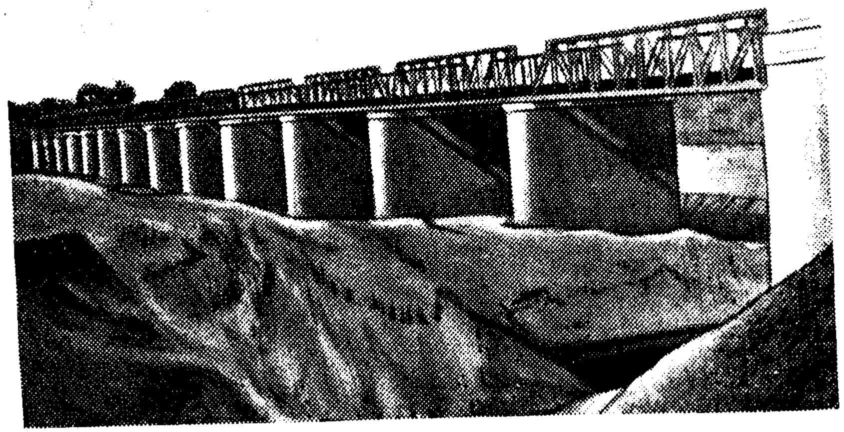
(4) Corrosion of concrete and reinforcement.

II. Current problems of concrete and reinforced concrete; prestressed concrete.

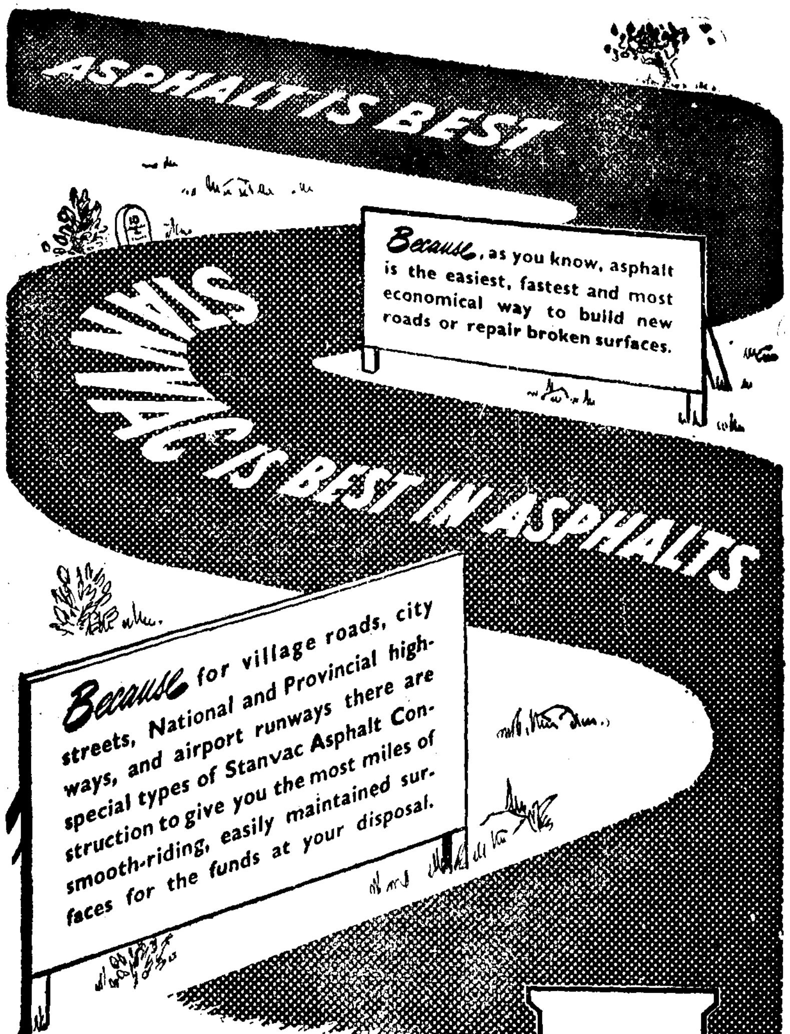
(1) Current problems of concrete and reinforced concrete.

(2) Progress in design and execution in connection with prestressed concrete.

(3) Dynamic stressing and fatigue strengths.



Gundalkamma Road Bridge, Guntur, 13 spans of 70-feet on Masonry Piers.



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SADABAHA BRIDGE: Having a total length of 270 feet this 13 feet wide bridge crosses the Sadabaha River in Bihar. The piers are of coursed rubble masonry in cement carried down in open foundations to rock. The bridge was completed in 1941.